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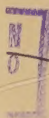


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PRACTICAL DESIGN OF SIMPLE
STEEL STRUCTURES

BY THE SAME AUTHOR

PRACTICAL DESIGN OF SIMPLE
STEEL STRUCTURES

VOLUME II.

GIRDERS, COLUMNS, TRUSSES, BRIDGES,
ETC.

INFLUENCE LINES

THEIR PRACTICAL USE IN BRIDGE
CALCULATION

PRACTICAL DESIGN OF SIMPLE STEEL STRUCTURES



VOL. I

SHOP PRACTICE, RIVETED CONNECTIONS
AND BEAMS, ETC.

WITH APPENDIX OF TABLES
(FORMERLY VOL. III)

A TEXT-BOOK

*SUITABLE FOR CIVIL ENGINEERS, STRUCTURAL
ENGINEERS, ROAD AND RAILWAY ENGINEERS, AND
STUDENTS AT UNIVERSITIES AND TECHNICAL COLLEGES*

BY

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FORMERLY A CIVIL ENGINEER WITH SIR WILLIAM ARROL & CO., LTD., BRIDGE BUILDERS, GLASGOW

FOURTH EDITION

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EXTRACT FROM THE PREFACE TO THE FIRST EDITION

THANKS to the many excellent books upon the Theory of Structures, the student has usually no great difficulty in calculating the primary stresses acting within a structure, but he often finds that these text-books afford very little guidance in the actual design and in the choice of sections. There are also several books of outstanding merit which give complete drawings of various structures, but these—although of great value to the more experienced designer—seldom explain why the particular sections used in the design were adopted in preference to others.

In the following pages the author has endeavoured to supply this want and also to provide a detailed guide for young engineers and architects who are entering the study of the Design of Structures. Special attention has been devoted to giving full details of each step in the process of making a complete design. Only a slight knowledge of the Theory of Structures is presumed, and none whatsoever of Sections or Shop Practice. As far as possible, an endeavour has been made to keep to the more elementary Mathematics, but it is well to warn the student that a good working knowledge of Mathematics and the principles of Mechanics is essential if he is to proceed to the study of more complicated structures.

Calculations are essential in designing, especially in structures, but it should be pointed out to beginners that figures must not be used too dogmatically in every case, since they often provide merely an indication of comparative values to the experienced designer. Practical knowledge is just as essential to a good design as ability to make calculations. The aim in designing is to make a structure which will be as far as is possible equally strong in all its parts, so that when purposely overloaded to cause failure each part will collapse simultaneously.

“ Have you heard of the wonderful one-hoss shay,
That was built in such a logical way
It ran for a hundred years to a day,
And then, of a sudden, it . . .
. . . went to pieces all at once,—
All at once, and NOTHING FIRST,—
Just as bubbles do when they burst.”

“ The Deacon’s Masterpiece,” by Dr. Oliver Wendell Holmes.

It is hoped this treatise will fill a niche amongst the other books on Structural Design and that it will be found useful to those for whom it is intended. The methods employed in the text are those used by the author while engaged in practice, modified by his experience in teaching students.

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ABBREVIATIONS AND SYMBOLS

The following symbols are also explained throughout the text.

A . . .	Area of one flange of a plate girder.	$U.D.L.$. . .	Uniformly distributed load.
A_c . . .	Ditto for compression flange.	$U.D.L.L.$. . .	Uniformly distributed live load.
A_t . . .	Ditto for tension flange.	V . . .	Vertical component.
B . . .	Bearing (of a rivet).	$V_L : V_R$. . .	Vertical reaction at left hand; ditto right hand.
$B.M.$. . .	Bending Moment.	W . . .	Load or web area, as per text.
$B.S.$. . .	British Standard.	Z . . .	Section modulus.
$C.G.$. . .	Centre of gravity.	b . . .	Breadth of a section.
C/C or C to C	Centre to centre.	d . . .	Diameter of a rivet or hole.
$C. of P.$. . .	Code of Practice, Bridges.	d . . .	Depth of a rectangle.
D . . .	Effective depth of girder.	e . . .	Eccentricity of load in inches.
$D.L.$. . .	Dead load.	f and F . . .	Actual and permissible stress in tons/sq. in.
$D.S.$. . .	Double shear.	f_c and F_c . . .	Ditto, compression.
E . . .	Young's modulus.	f_t and F_t . . .	Ditto, tension.
$E.U.D.L.$. . .	Equivalent uniformly distributed load.	f_u and F_u . . .	Ditto, shear on web pls.
F . . .	Shearing force.	f_b and F_b . . .	Ditto, bearing on rivets.
G . . .	The first moment.	f_{ds} and F_{ds} . . .	Ditto, double shear on rivets.
H . . .	Horizontal force.	f_s and F_s . . .	Ditto, single shear on rivets.
I . . .	Impact coefficient or moment of inertia, as per text.	ft. or ' . . .	Feet.
I_x . . .	Moment of inertia about horizontal axis.	ft. T or "T . . .	Foot tons.
I_y . . .	Ditto, vertical axis.	k . . .	Radius of gyration.
L . . .	Span of girder.	l . . .	Span or length in inches.
$L.L.$. . .	Live load.	l/k . . .	Slenderness ratio.
M . . .	Moment.	p . . .	Rivet pitch.
$M_A ; M_B$. . .	Moment at A ; at B .	q . . .	Intensity of shear stress.
Max. . .	Maximum.	t . . .	Plate thickness in inches.
$M.R.$. . .	Moment of resistance.	y . . .	Distance to outer fibres from the N.A.
N.A. . .	Neutral axis.	Δ . . .	Deflection (Delta).
P . . .	Force, generally a pull.	$\triangle$. . .	Triangle.
Pl. . .	Plate.	ϕ . . .	Diameter of a rivet or hole.
R . . .	Rivet value : least in $S.S.$, $D.S.$, or B .	θ, ϕ . . .	Angles (theta and phi).
R_A . . .	Reaction at A .	$\hat{A}BC$. . .	Angle ABC.
R.S.J. . .	Rolled steel joist.	$<$. . .	Less than.
S . . .	Shear or stress, as per text.	$>$. . .	Greater than.
$S.S.$. . .	Single shear.	$\nless$. . .	Not less than.
T . . .	Tee bars.	Σ . . .	The sum of (Sigma).
T . . .	Superscript, small capital = tons (British or "long" ton of 2,240 lb.).	$=$. . .	Equal to.
$U.D.D.L.$. . .	Uniformly distributed dead load.	$\angle$. . .	Angle section.

PERMISSIBLE STRESSES ON MILD STEEL* FOR BUILDINGS

		Stress Symbol.		Tons per sq. in.
		Actual.	Permissible.	
TENSION.	Axial stress on net section	f_t	F_t	9
COMPRESSION.	Columns. See tables	f_c	F_c	
BENDING.	Beams. Extreme fibre stress ; tension	f_t	F_t	10
	compression, either	f_c	F_c	10
	or (lesser of)	"	"	$\frac{1,000}{l/k} K_1$
	where l = unsupported laterallength. k = rad. of gyrat., axis yy . K_1 = unity, except for R.S.J. and plate girders symmetrical about both principal axes when it has the following values k_x/k_y 5 $4\frac{1}{2}$ 4 $3\frac{1}{2}$ 3 or less K_1 1 $1\frac{1}{8}$ $1\frac{1}{4}$ $1\frac{3}{8}$ $1\frac{1}{2}$ Plate Girders. The above per- missible values F_t and F_c shall be reduced by 5 per cent., i.e., 9.5, 9.5, etc.			
SHEAR.	Stress on gross cross-section of webs, (the lesser of) either or $\left(\frac{225}{b/t}\right)^2 \left[1 + \frac{3}{4} \left(\frac{b}{a}\right)^2\right]$ where a and b = greater and lesser unsupported dimensions of a web panel, respectively. t = web thickness.	f_w	F_w	6.5
BEARING.	Steel on steel	f_b	F_b	12
RIVETS.	See opposite page.			
BOLTS.	" " "			

* Based upon the British Standard Specification, No. 449.

FOR WORKSHOPS AND STEEL FRAMED BUILDINGS

	Dia. of hole in inches.	Shear value in tons per rivet.		Bearing value in tons per rivet for a plate thickness (in inches) of †										Axial tension in tons. Value of one Rivet. Bolt.*		Tons per sq. inch.
		single	double	1	1/8	3/16	1/4	5/16	3/8	1/2	5/8	3/4	3/4	1/2		
				1-18	2-36	1-50	1-88	2-25	2-63	3-28	3-94	4-50			5-06	
SHOP RIVETS AND T.F. TURNED BOLTS	1/2	0-1963	1-18	2-36	1-50	1-88	2-25	2-63	3-28	3-94	4-50	5-06	5-63	0-98	0-61	6
	3/8	0-3068	1-84	3-08	1-88	2-34	2-81	3-38	3-94	4-27	4-88	5-48	6-10	1-53	1-02	12
	1/2	0-4418	2-65	5-30	2-25	2-81	3-05	3-66	4-27	4-88	5-48	6-10	6-70	2-21	1-82	12
	3/4	0-5185	3-11	6-23	2-44	3-05	3-28	3-94	4-59	5-25	5-91	6-56	7-22	2-59	2-20	12
	1	0-6013	3-61	7-21	2-63	3-28	3-94	4-59	5-25	5-91	6-56	7-22	7-83	3-01	2-53	5
	1 1/4	0-6903	4-14	8-28	2-81	3-52	4-22	4-92	5-63	6-32	7-03	7-73	8-44	3-45	2-98	5
	1 1/2	0-7854	4-71	9-42	3-00	3-75	4-50	5-25	6-00	6-75	7-50	8-25	9-00	3-93	3-32	6
SITE RIVETS	1/2	0-1963	0-98	1-96	1-25	1-56	1-88	2-19	2-73	3-13	3-75	4-22	4-69	0-79	—	5
	3/8	0-3068	1-53	3-07	1-56	1-95	2-34	2-73	3-13	3-55	4-06	4-57	5-08	1-23	—	10
	1/2	0-4418	2-21	4-42	1-88	2-34	2-81	3-28	3-75	4-22	4-69	5-16	5-63	1-77	—	10
	3/4	0-5185	2-60	5-19	2-03	2-54	3-05	3-55	4-06	4-57	5-08	5-58	6-02	2-07	—	4
	1	0-6013	3-01	6-01	2-19	2-73	3-28	3-83	4-38	4-92	5-47	6-02	6-56	2-41	—	4
	1 1/4	0-6903	3-45	6-90	2-34	2-93	3-52	4-10	4-69	5-27	5-86	6-45	7-03	2-76	—	4
	1 1/2	0-7854	3-93	7-85	2-50	3-13	3-75	4-38	5-00	5-63	6-25	6-88	7-50	3-14	—	4
BLACK BOLTS	1/2	0-1963	0-79	1-57	1-00	1-25	1-50	1-75	2-19	2-50	3-00	3-38	3-75	—	0-61	4
	3/8	0-3068	1-23	2-45	1-25	1-56	1-88	2-19	2-50	3-00	3-38	3-75	4-16	—	1-02	8
	1/2	0-4418	1-77	3-53	1-50	1-88	2-25	2-63	3-00	3-38	3-75	4-16	4-56	—	1-82	8
	3/4	0-5185	2-07	4-15	1-63	2-03	2-44	2-84	3-25	3-66	4-06	4-46	4-81	—	2-20	5
	1	0-6013	2-41	4-81	1-75	2-19	2-63	3-06	3-50	3-94	4-38	4-81	5-25	—	2-53	6
	1 1/4	0-6903	2-76	5-52	1-88	2-34	2-81	3-28	3-75	4-22	4-69	5-16	5-62	—	2-98	5
	1 1/2	0-7854	3-14	6-28	2-00	2-50	3-00	3-50	4-00	4-50	5-00	5-50	6-00	—	3-32	6

* Axial stress on the area at the bottom of the bolt thread.

† The bearing values to the right of the upper zigzag line are greater than the corresponding double shear value.

PRACTICAL DESIGN OF SIMPLE STEEL STRUCTURES

CHAPTER I

ROLLED SECTIONS FOR STRUCTURAL PURPOSES

Sections. Before entering into the discussion of details a short discourse upon the sections used in constructional work may prove helpful, since successful designing depends upon many things, not least of which is the choice of suitable sections.

The white-hot and plastic ingot of steel, known as a bloom, is reduced either under a steam hammer or in a cogging mill. From there it is taken to the finishing rolls. These may be likened to huge mangles with a pair of horizontal steel or iron rollers, one mounted above the other.

Plates are formed by passing and repassing the metal through the rolls, which are made to approach each other previous to each successive rolling, until the desired thickness of metal is obtained.

In the case of other sections the cylindrical rolls have circumferential grooves or chases let into their surfaces. The metal when rolled is crushed into this chase and emerges at the other side a different shape from what it had at entry. An originally square bloom after passing through a varying set of chases finally comes forth as an angle bar or some other type of section, depending upon the outlines cut into the rolls. The finishing roll of an angle mill is given in Fig. 1.

Steel section rollers have their names cut into the rolls, and the finished section bears, in raised letters, the maker's name and sometimes the size and weight in pounds per foot length of the section. There is in addition, according to the British Standard Specification No. 15, For "Structural Steel," a mark which enables the finished steel to be traced to the original cast.

Continental steel sections are usually cheaper than British sections, but the home article is rather more dependable, and is distinguishable from the foreign material by reason of the maker's name, which is rolled on the section. With the exception of American sections,

foreign sections are usually rolled to metric dimensions. These centesimal dimensions have to be converted into inches, and on adding up dimensions the odd decimals of an inch are at times rather troublesome.

Sectional Growth. After constant use the chases in the rolls become enlarged and the finished article has a slightly larger cross-sectional area than that turned out when the rolls were new. In consequence of this steel manufacturers claim a ROLLING MARGIN of $2\frac{1}{2}$ per cent. over or under the listed weight per foot. The custom of paying the mills for the actual weights passed over the weighbridge has caused the sections to be rolled slightly on the heavy side. It is preferable that these should be over rather than under the weights listed, as the area of metal allowed for in the design is thereby assured. Any excess over the $2\frac{1}{2}$ per cent. margin is not paid for by the receiver. The flanges of a joist or the legs of an angle are

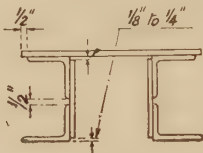
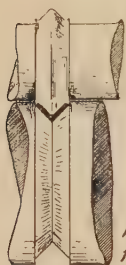


FIG. 1

FIG. 2



FIG. 3

*Finishing Rolls
for an Angle Bar.*

often slightly wider than the listed dimensions, due to the aforementioned wearing of the rolls, and for this reason it is wise never to butt the edges of sections together when detailing a drawing.

The arrangement of the component parts of the section of Fig. 2 permits slight increases in the sizes of the individual members without departing from the general outline of the built-up section. In Fig. 3 everything is shown as tightly butted, and no allowance is made for sectional growth, with the result that when actually assembled the continuity of outline is broken. Angles will project out past plates, or *vice versâ*.

PLATES (abbreviation "Pl.")

Plates may be of any size or thickness provided they do not exceed the maxima given in Table 6, Appendix. The maker can deliver a maximum area of plate of a given thickness, but if the width of the plate is large, then the length of the plate must be correspondingly shorter, or *vice versâ*; the product of the length and the breadth must not exceed the maximum area.

Plates should always be specified as breadth by thickness by length, *e.g.*, 2 Pls. 16" $\times$ $\frac{3}{8}$ " $\times$ 9' 0". The breadth, be it noted, is in inches, and not in feet and inches, as this facilitates the "running out of the weights" when estimating and diminishes the labour of collecting the plate sizes when ordering the material. In spite of the advantages accruing from this method, it is not uncommon to find the plates specified as 2 Pls. 9' 0" $\times$ 1' 4" $\times$ $\frac{3}{8}$ ". The beginner is prone to be slipshod in little things like this until he appreciates that these trifles when properly systematized mean a saving of time and money.

Plates which have to be planed on both long edges should be ordered $\frac{1}{4}$ in. or $\frac{3}{8}$ in. wider than the finished width. This permits $\frac{1}{8}$ in. or $\frac{3}{16}$ in. being planed off each side.

It will be observed from Table 6, Appendix, that plates varying by $\frac{1}{32}$ in. are avoided in structural work, and only plates whose thicknesses are some multiple of $\frac{1}{16}$ in. are used. One can differentiate at a glance between a $\frac{5}{8}$ in. and an $\frac{11}{16}$ in. plate, but more care must be exercised when choosing between two plates whose thicknesses are $\frac{3}{32}$ in. and $\frac{11}{16}$ in. respectively.

Large plates have a tendency to arrive in the yard from the mills slightly thicker in the centre than round the edges. This seldom has any significance in practice.

Plate Extras. Plates larger than those listed in the table are rolled, but are subject to special arrangements and prices.

Material which is thinner than $\frac{3}{16}$ in. is designated a sheet, and the rolling margin (of 5 per cent., *i.e.*, $2\frac{1}{2}$ per cent. under or over) is increased to 10 per cent., *i.e.*, 5 per cent. under or over.

Narrow plates cannot be sheared exactly parallel-sided; the curve varies more or less according to the length and thickness and the ordinary rolling margin is not applied.

Sketch Plates. Extras are charged on all plates whose shapes necessitate more than four cuts with the shears, *i.e.*, other than rectangular plates. If there is an extra for area, the tapered or sketch plate is charged upon the area of the parent plate from which it was cut.

Chequered Plates are plates which have a raised diamond pattern on their surface after the manner of Fig. 4. Their thickness is generally "measured on the plain, *i.e.*, under cheque." They prove extremely useful as flooring where light loads have to be carried, such as on access gangways and machine pit

Length way & direction of pattern.



ADMIRALTY DIAMOND CHEQUERED PLATE —

FIG. 4

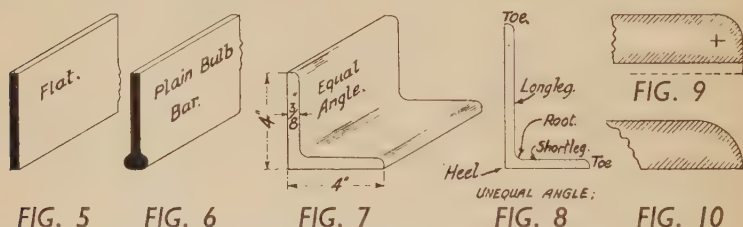
covers. In addition they are easily lifted and replaced, while the chequered surface presents a foothold not offered by an ordinary plate, especially where oil is apt to be spilled.

FLATS (Symbol —)

The lower of the two rolls, through which the bar finally passes, has flanges which give the bar a definite width ; *vide* Fig. 5.

If ordered to the correct width, a flat needs no planing to true it up. However, should the flat be ordered long and narrow, it will possibly not be rectangular, but will be slightly curved in its length.

Flats are slightly dearer than plates, but as their proper use saves handling and edge planing, they probably come out cheaper. The quantity and the job on hand will settle whether they should be used or not. The corner edges of flats are often a trifle rounded and not



clean-cut ; they thus have the disadvantage that when piled together to form the flange of a plate girder they do not present the same perfect finish as machined plates. Their use in this case, therefore, depends upon the engineer's specification and whether the girder is exposed to near view or not. If the girder is small and only one flange plate is required, the ideal section to use is the flat. Should the flange plate require to be spliced, and if the girder is in near view, use plates, as more perfect alignment is obtainable.

Maximal Sizes to which flats are rolled are listed in Table 7, Appendix.

Bulb Bars. A bulb bar is a flat with a bulbous edge (Fig. 6). The bulb portion stiffens the flat should it be in compression, and the line of rivets rather far removed from the free edge of the flat. This section is more used in shipbuilding than in bridge and shop construction.

ANGLES (Symbols, $\angle$ for equal legs and $\angle$ for unequal legs, Figs. 7 and 8)

This particular section is probably the most useful section rolled. The thickness of each leg is the same no matter whether the angle

is equal or unequal. An angle is always specified thus: $1 \angle 4" \times 3" \times \frac{3}{8}" \times 10' 0"$, which reads one angle measuring 4 in. by 3 in. by $\frac{3}{8}$ in. thick by 10 ft. long. In the case of an unequal angle the longer leg is given first. The weight of an ordinary angle is never mentioned on drawings; the sizes of the legs and the thickness provide sufficient information.

A practice, fortunately rare, is to specify an equal angle of, say, $4" \times 4" \times \frac{3}{8}"$ as $4" \times \frac{3}{8}"$. The saving of time thus effected is negligible, while the resulting mistakes through misinterpretation are certainly not.

The toe of the angle is shown to a larger scale in Fig. 9. From the position of the centre (obtained from Table 1, Appendix) of the segmental outline it is obvious that Fig. 10, which is the beginner's usual conception, is wrong. The root radius is larger than the toe radius, but these are constant for different thicknesses of the same angle. Further, all angles the sums of whose flanges or legs give the

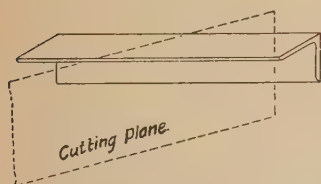


FIG. 11

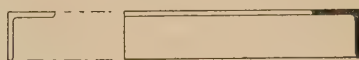


FIG. 12

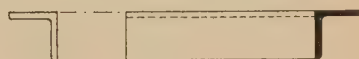


FIG. 13

same total have similar root and toe radii; *e.g.*, for a $3\frac{1}{2}" \times 3\frac{1}{2}" \times \frac{3}{8}"$ and a $4" \times 3" \times \frac{1}{2}"$ angle the total is 7 united inches.

To facilitate the reading of a drawing it is often advisable to indicate a break in the section. The broken surface is generally filled in with ink to emphasize the outline. The end view of an angle or any other section is not ink-filled. Fig. 11 demonstrates how the imaginary cutting plane should always be on the skew, while Figs. 12 and 13 show the resulting effects upon an angle.

Length of Angles. Long angles are unwieldy to work with in the shops, and, moreover, if of light section, are apt to arrive there badly twisted in shipment from the mills. Wherever possible lengths should be specified upon which there is no extra to pay.

Stockyards (*i.e.*, people who buy in quantity from the mills to supply small demands) generally stock most sections up to 40 ft. in length, which forms quite a fair maximum to work.

Thickness of Angles increases by $\frac{1}{16}$ in., although many steel merchant tables only give the list of heavier sections as increasing

by $\frac{1}{8}$ in. The various properties of the $\frac{1}{16}$ in. increase of thickness can be closely arrived at by interpolation.

Thick Angles. Angles can be rolled to a greater thickness than the maximum specified in the tables, and can be readily obtained if the tonnage order is sufficiently large. The extra thickness of metal is obtained by opening out the rolls.

As Fig. 14 shows, the heel of the bar is still sharp in profile, but the toes are slightly rounded on the outside; this will be understood by reconsulting the diagrammatic elevation of the finishing rolls in Fig. 1.

Thick angles generally prove themselves troublesome in constructional steelwork, especially so if they are long and require to be spliced. The thick leg and sometimes thicker cover leave very little room for the rivets in the adjoining leg. It is impossible to use an internal cover in the case of the $3" \times 3" \times \frac{5}{8}"$ angle shown in the dotted outline of Fig. 15, as dimension B is too small,



FIG. 14

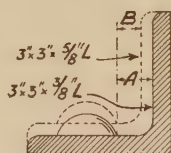


FIG. 15

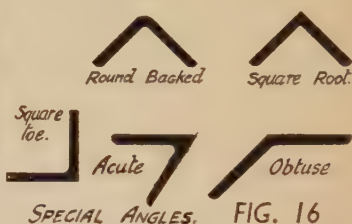


FIG. 16

whereas dimension A , of the $3" \times 3" \times \frac{3}{8}"$ angle (hatched cross-section), permits both the use of an internal cover and the closing of the $\frac{3}{4}$ inch diameter rivet in its standard position. Both angles are British Standard equal angles.

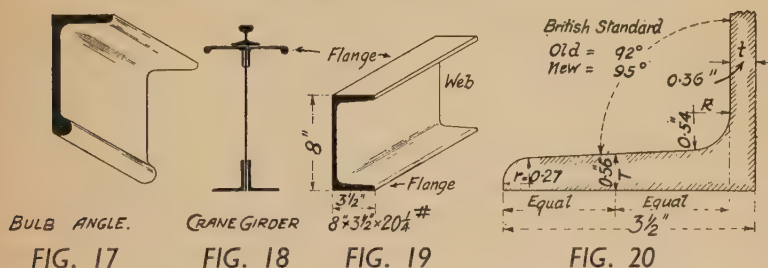
Weight of Steel is taken at 489.6 lb. per cubic foot. From this an extremely useful figure is obtained for the running out of weights; viz., a bar $1" \times 1"$ in cross-section weighs 3.4 lb. per foot run.

A rapid and much-used rough approximation is to take the cross-sectional area and mentally shift the decimal point one place to the right (i.e., multiply by 10) and then divide by 3. The error is $3.4 - 3.33 = 0.07$ lb. per square inch per foot run.

Area and Weight of Angles. The cross-sectional area of an angle is made up of two rectangles, which in the case of Fig. 7 are $4" \times \frac{3}{8}"$ and $3\frac{5}{8}" \times \frac{3}{8}"$. The cross-sectional area is therefore $7\frac{5}{8}" \times \frac{3}{8}" = 2.86$ sq. in., the extra metal at the root fillet balancing the loss at the toes. The weight per foot $= 2.86 \times 3.4 = 9.72$ lb., which is the listed weight. Using the approximate method, the weight per foot $= 28.6 \div 3 = 9.54$ lb. Tables giving united inches and

weights are based upon the preceding rule of rectangles. If two or more angles (and tees) have the same thickness, their weights and areas are similar, provided that the flange sums (in united inches) are the same. The following angles have the same sectional area : $4'' \times 4'' \times \frac{3}{8}''$, $5'' \times 3'' \times \frac{3}{8}''$ and $4\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$, since the united inches are 8.

Special Angles. The cross-sections of five special angles are illustrated in Fig. 16. These naturally are more difficult to obtain, and may cost anything from a few shillings to several pounds more per ton over the ordinary angle basis price. An immediate consequence of this is that where a small length of a special angle is indicated an ordinary angle is planed or smithed to suit. A common feature to find on drawings is a round-backed angle serving as an internal cover at an angle splice. Instead of ordering this angle, the constructional works plane the heel away from another angle,



in order that the angle cover may fit into the root of the main angle ; alternatively a plate of the specified thickness may be bent to suit. See p. 83.

Again, should a drawing demand a $2\frac{3}{4}'' \times 2\frac{3}{4}'' \times \frac{3}{8}''$ angle (which is a British Standard section), and inquiry prove that it may be several weeks before this section is obtainable, the constructional works, instead of waiting, may replace the specified section by a $3'' \times 3'' \times \frac{3}{8}''$ angle. Permission is generally sought to thus replace a section. On the other hand, the $2\frac{3}{4}'' \times 2\frac{3}{4}'' \times \frac{3}{8}''$ size of angle may be imperative if there is not sufficient clearance to use the larger section. Provided the quantity is not excessive, the usual procedure is to plane away the toes of a $3'' \times 3'' \times \frac{3}{8}''$ angle and reduce it to a square-toed angle (Fig. 16) measuring $2\frac{3}{4}'' \times 2\frac{3}{4}'' \times \frac{3}{8}''$. The root radius of the "manufactured" angle is slightly larger than that of the angle specified, but that is of no importance whatsoever. A considerable saving in time is effected by the operation described at the cost of a few pence per lineal foot of angle.

Bulb Angles. Due to their being used in shipbuilding, these

sections can be much more readily obtained than the special angles mentioned above. Certain scantlings (*i.e.*, sizes) are at the ordinary angle basis price. For outline see Fig. 17.

The bulb helps to stiffen the outstanding leg, especially when the angle is under compression along its length. The small crane girder of Fig. 18 has the upper, or compression, flange formed of two of these bulb angles. A small depth horizontal girder, whose flanges are the bulbs, is thus formed to counteract the bending action which sometimes occurs when the craneman, instead of using a direct lift and cross travel, drags the load across the shop floor in a direction normal to the girder length.

CHANNELS (Symbol $\square$)

The channel section is an extremely useful one in structural design. Channels are listed as depth by flange width by weight in pounds per foot run. When the weight comes to an odd decimal the nearest quarter-pound is often quoted on the drawings. The 5 per cent. rolling margin is sufficient justification for this practice. Fig. 19 gives a cross-section of an $8'' \times 3\frac{1}{2}'' \times 20\frac{1}{4}$ lb. channel; as listed it weighs 20.21 lb. per foot.

One flange of a $10'' \times 3\frac{1}{2}'' \times 24\frac{1}{2}$ lb. channel (24.46 lb.) is drawn to a large scale in Fig. 20. Angles have parallel-faced legs, but the channel has the inner face splayed at an angle of 95 degrees to the web. In no case is the web thicker than the flange. The thickness of the latter is measured at a point half-way between the inner web face and the flange toe, and not midway between heel and toe. Various steel merchants' catalogues usually assign the symbols shown to the various radii and thicknesses.

A draughtsman's "trade trick" is to mark his clinograph or adjustable set square with two incisions, as in Fig. 21. By adjusting the outer edge of the swing arm, co-linear with the knife marks, the splays of joists and channels are at once obtained. This, even in the course of a year, means a considerable saving of time. Others, again, plane down old or broken celluloid set squares to the requisite bevel.

The author incorporated this idea in the "C. E. Adjustable Set Square," which, having no projecting screws, can be worked with either face in contact with the drawing paper. The end of the moveable arm slides within a slot in the segmental inner edge of the vertical arm, which has degrees, half degrees and slopes of channel and rolled steel joist flanges marked on it. This celluloid instrument is therefore also equivalent to a $10\frac{1}{2}$ in. diameter protractor.

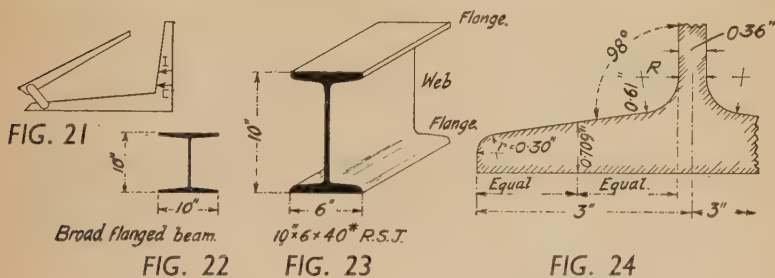
Thick Channels. By opening out the rolls a heavier scantling of channel is obtainable. The extra metal is given along the back or heel side of the web, so that the web is thickened up and is suitable for use with heavy shearing forces, where depth of headroom is a vital consideration. The flanges are therefore slightly lengthened along the outer faces, but otherwise unaltered.

Special Channels are obtainable, but are seldom used in constructional steelwork.

JOISTS (Symbols I and H)

Joists are also known as beams, I beams, H beams, or as R.S.J.'s, the usual contraction for rolled steel joists.

This section with channels and angles form the triumvirate of sections in greatest demand. The joist is especially favoured by



architects and builders to carry floors, door lintels, stair treads, etc., and is more used by them than any other steel section in ordinary brick or stone construction.

R.S.J.'s are specified in a similar manner to channels, viz., depth by flange width by the weight per foot in pounds; see Fig. 23. Unlike channels, however, the listed weight is without decimals. The set of the inner face of the joist flange is 98 degrees to the web, and is constant for the various British Standard beams (Fig. 24).

Thick R.S.J.'s can be obtained by lifting the rolls, a practice which is not nearly so common as with the previous sections. The extra metal is added centrally along the web. The flanges are unaltered except that they are wider by the amount of the web's increase of thickness.

Special R.S.J.'s. Broad flange beams (Fig. 22) are rolled beams with exceptionally wide flanges. They are not British Standard joists, and, although rolled abroad, can be obtained in that quality of steel which meets the requirements of the British Standard

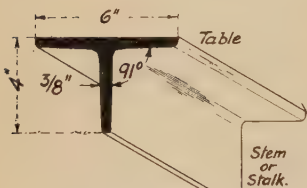
Specification. In some sections the flanges are not tapered, but are parallel, while the toes are square.

A few sections come from America, but the majority of them are of Continental origin. A broad flange beam may be listed as a $10'' \times 10''$, whereas it really measures $9\frac{2}{3}\frac{7}{8}'' \times 9\frac{2}{3}\frac{7}{8}''$ (i.e., 9.843 in.), the metric units being 25 cm. $\times$ 25 cm. Under certain conditions they possess slight advantages over the British Standard "B" heavy beams or pillars, and their popularity would doubtless grow were they rolled in this country.

TEES (Symbol T)

Tee bars are specified by giving the table dimension by the stem dimension by the thickness, as in Fig. 25.

The most useful sections are the smaller sizes of tees which are



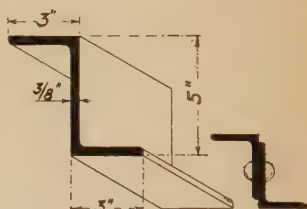
6" $\times$ 4" $\times$ $\frac{3}{8}$ " TEE BAR.

FIG. 25



BULB TEE.

FIG. 26



ZED BAR.

FIG. 27



FIG. 28

used for astragals in ordinary putty glazing for side and roof lights. These sections are $1\frac{1}{2}'' \times 1\frac{1}{2}'' \times \frac{1}{4}''$, $1\frac{3}{4}'' \times 1\frac{3}{4}'' \times \frac{1}{4}''$ and $1\frac{1}{2}'' \times 2'' \times \frac{1}{4}''$.

At one time tees were greatly used for the main rafters and struts of roof trusses, but their employment entailed eccentric riveted gusset connections, which could only be eliminated by cutting away either the stem or the table. In the smaller sections little clearance was obtainable for riveting. Modern practice with riveted work has forsaken T-bar rafters for two angles back to back.

Weight of T-bars. Angle bars and tee bars have the same weight per foot approximately if their united inches and thicknesses are the same, viz., the $6'' \times 4'' \times \frac{3}{8}''$ tee of Fig. 25 has the same weight per foot as a $6'' \times 4'' \times \frac{3}{8}''$ or a $5'' \times 5'' \times \frac{3}{8}''$ angle, the united inches being 10.

Special T-bars are rolled for several patent glazing firms, the rolls having been cut to their requirements. These special tees are sometimes covered with a metallic covering of lead or tinned lead.

Bulb Tees (Fig. 26) are rarely found in shop or bridge construction.

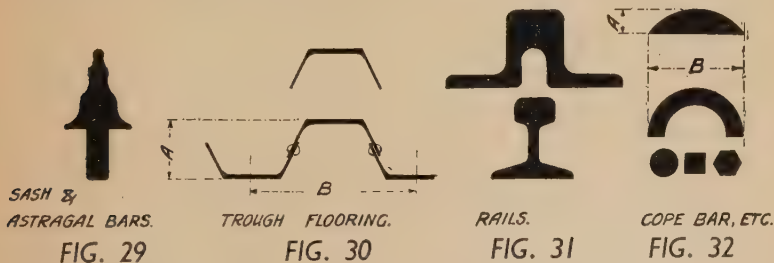
ZED BARS (Symbol Z)

Zed bars are specified as depth by flange by thickness; Fig. 27 therefore shows a $5" \times 3" \times \frac{3}{8}"$ zed bar. Their use in this country is practically "tabooed" by several constructional firms, whereas in the United States the reverse is the case. The former firms would apparently prefer to spend money on drilling and riveting angles, after the manner of Fig. 28, rather than use rolled zed bars. Against the cost of workmanship must be placed the saving in the price of raw material, since zed bars cost a little more per ton than angle bars. Further, angles are commoner sections than zed bars, so the final built-up zed shape is more quickly obtained, with the additional advantage of a great range of section depth and section modulus. The zed type of section, rolled or built, is mostly used for purlins.

MISCELLANEOUS ROLLINGS

Fig. 29. Sash bars and astragals have many varied forms, as will be found from the catalogues of the firms who specialize in this work.

Fig. 30. The upper diagram is a cross-section of a rolled steel trough employed in flooring, especially for bridges. The lower diagram indicates how a continuous width is obtained. Previous to the ballast being laid, the floor is brought up to a level surface by filling in with concrete, the matrix of which is generally of portland cement, or, alternatively, of a bituminous material such as tar, asphalt, etc. With pressed steel flooring the buckles or corrugations are pressed out of heated steel plates, and of necessity are of the same thickness throughout. Both the furnace and the press must be sufficiently large to take the plate at one operation



The latter form of flooring offers a larger field for personal ideas than rolled troughing, and for small quantities of special outline is considerably cheaper.

Sometimes, however, troughing is pressed cold from a special quality of steel, which should have a tensile breaking strength of 25 to 30 tons per sq. in. with an elongation of not less than 23 per cent. on the standard 8 in. gauge length—as against 28 to 33 tons and an elongation of 20 per cent. for standard quality steel. If the cold pressed plates are $\frac{1}{2}$ in. or more in thickness the troughing must be annealed after leaving the press.

The particular type illustrated can be had in different sizes, *viz.*, A by B , varying from $4'' \times 12''$ to $1' 1'' \times 2' 8''$; the respective weights per square foot of covered area are 13.4 lb. and 28.8 lb., and the section moduli are 4.9 in. and 72.7 in. cubed.

Fig. 31. In addition to the bridge rail and flat bottom rail which are illustrated, there are the tram rail, the heavy crane rail, and the bull-headed rail. Nearly every railway has its own particular type of the latter form of rail. Old and worn rail sections prove extremely useful as reinforcement for grillage work in concrete foundations.

Fig. 32. Cope bars and hollow half-rounds are useful in making hand railings. Cope bar makes an ideal attachment for corrugated sheeting. See Plate II., Vol. II. Dimensions $A \times B$ can be had from $\frac{1}{4}'' \times \frac{3}{4}''$ up to $1\frac{1}{2}'' \times 7''$.

Round Bars. In the basements of shops and warehouses, where space is extremely valuable, steel columns are made of large diameter solid steel round bars.

The caps and bases are also of solid steel and have holes bored out of them slightly less in diameter than the finished diameter of the turned ends of the shaft.

Both cap and base are then heated and fitted on to the shaft, which they tightly grip on cooling and shrinking. If not properly aligned, the load may act eccentrically in the column, causing a bending stress as well as a direct thrust. Only in exceptional circumstances is such an uneconomical column used.

Other Extras on Sections. The more work which is expended on the rolled sections at the mills over and above the usual mill finish the greater is the extra tonnage rate.

Cold-straightening is included in the basis price of rolled steel joists, but is an extra on all other sections. Constructional works always cold-straighten sections which may arrive slightly twisted, otherwise holes would not be in register.

Oiling, Painting and Annealing must be paid for if done at the mills.

Tests and Tensile Strength, etc. Any departure from the standard routine and specification, which involves special attention at the steel company's works, produces a corresponding extra.

Cutting bars to length is done when the bars are hot to a margin

of 1 in. under or over the specified lengths. Alternatively the manufacturer may be given a 2 in. cutting margin over the specified length, but nothing under this length.

Exact Length. Cutting by hot saw prevents accuracy of marking off lengths, so that if exact lengths are ordered the bars are cold-sawn. Manufacturers define the word "exact" as meaning to within the nearest $\frac{1}{8}$ in. over or under; others, again, quote $\frac{1}{16}$ in. over or under. Extras are charged by the mills for this. The constructional works do this exact cutting to length themselves, and in consequence seldom order anything from the mills other than with the usual mill finish.

Common Sections. Although an angle or any section is given by the British Standards Institution in their list, it does not necessarily follow that this section can be easily obtained. Practice has found several particular sections extremely useful, with the result that a demand is created for these sections. The steel-makers concentrate on these sections to the detriment of others on their list. Some sections may thus be rolled once a week, or once a month, or perhaps only once a year.

The cost entailed in changing and setting up the rolls can only be justified by a sufficiently large rolling of the new section. Again, it might even be worth spending several thousand pounds in getting special heavy rolls made and turned provided that the rolling is sufficiently large. The larger the rolling the less is the cost of the rolls per ton of output. To balance this cost, there is probably a saving in drilling, riveting, planing, etc., of a built-up section. In the case of the strengthening of the floor of the Forth Bridge (*Minutes of the Proceedings Inst. C.E.*, Vol. CCXV., 1922-23, by W. A. Fraser, M.Inst.C.E.) a rail trough composed of three rolled steel channels was adopted. These channels, which are 17 in. deep, had to be specially rolled, but the resulting trough is an extremely rigid one, and was obtained with the minimum amount of labour. Again, patent glazing companies have rolls especially cut to roll their astragal bars, not a costly undertaking with light sections. The heavier the section the greater the cost of cutting the rolls and changing them in the mills.

Choice of Sections. The simplest rule for the embryo designer is to choose sections which can be readily obtained. As a help in this direction the sections given in the tables of the Appendix have been arranged so that no difficulty should be experienced in picking out the commoner sections. With the above rule may be coupled the remark that, broadly speaking, angles, joists and channels generally prove themselves the cheapest and most efficient of all sections.

REFERENCES

Steel Rolling Mills. *Vide the Journals of the Iron and Steel Institute*, which give descriptions of various types of mills, gearing, power used, practice and methods, etc. The index volumes of the Institute give a comprehensive list of other publications dealing with this subject.

CHARNOCK, G. F. *Mechanical Technology* (Constable & Co. Ltd.).

Standard Sections. *Dimensions and Properties of British Standard Rolled Steel Sections for Structural Purposes*, issued by the British Standards Institution of 28, Victoria Street, London. This book is the source of the properties list of standard sections given by the various steel companies. The books issued by these companies include many sections rolled by them which are not standard, but nevertheless useful.

Structural Steel. *British Standard No. 15.* Specifies :—Process of manufacture, contents, properties, rolling and cutting margins, etc.

Section Books. The following books give a list of the properties of sections in addition to much other useful information ; in fact, in one or two cases these books approach the text-book standard, so copious are the notes and data. They may usually be obtained free on payment of postage, but much depends upon the position in business occupied by the applicant. This list is by no means a complete one :—

Appleby-Frodingham Steel Co., Ltd., Scunthorpe, Lincolnshire.

Consett Iron Co., Ltd., Consett, Co. Durham (chequered plating, etc.).

David Colville & Sons, Ltd., Motherwell, Scotland.

Dorman, Long & Co., Ltd., Middlesbrough, England.

Redpath, Brown & Co., Ltd., Edinburgh, Glasgow, London and Manchester.

Continental and British sections, see *Structural Steel*, the handbook of Messrs. R. A. Skelton & Co., Moorgate Station Chambers, London, E.C. (on sale).

American sections, see *American Civil Engineers' Handbook* (Wiley & Sons) ; *Structural Engineers' Handbook*. Ketchum (McGraw-Hill).

CHAPTER II

THE DRAWING OFFICE

Tenders and Estimates. In point of fact, the original designer is the client, whether or not he possesses any knowledge of stresses and strains. The client desires a footbridge or some other structure erected to fulfil a number of conditions. It has to be a certain length, a certain width, a certain height, and so on ; money may or may not be spent profusely upon it, and its useful life may be fixed or indefinite. The client then turns to an architect, a consulting engineer, or sometimes directly to the constructional works, whomsoever the client may deem best fitted for the particular job on hand.

The job is now considered in more detail : loads are calculated, resulting forces found, and the sections finally settled. A design drawing is now made, usually to a small scale ($\frac{1}{4}$ in. to $\frac{3}{8}$ in. equal to 1 ft., depending upon the size of the structure), but all important connections and details are drawn separately to a larger scale (from $\frac{3}{4}$ in. to $1\frac{1}{2}$ in. equal to 1 ft.) to ensure that the preliminary design is feasible.

Very often at this stage consulting engineers and architects have the drawings traced in ink upon tracing cloth or paper. Blue prints are made and sent to the steel constructional works, together with a specification detailing the quality of the materials and workmanship, while the covering letter asks for a price to be tendered before a certain date.

Specifications. The best method of specifying is to state " that the work shall comply in all respects with the requirements of the (appropriate) British Standard Specification " and to amplify it by an accompanying enclosure. The latter should state which of the alternative courses or methods laid down in the standard specification shall be followed. This standard specification is eminently just to manufacturer and purchaser alike.

Special clauses are sometimes inserted which, if fully carried out, would virtually call for as much meticulous care and refinement in workmanship as is required in the manufacture of some delicate piece of mechanism. This can, of course, be done, but at an enhanced cost ; the work can be carried out just as well and efficiently without such unnecessary refinement. What is wanted

is an honest, straightforward job, such as is called for in the standard specification.

Time Clauses binding the delivery or completion down to a certain day are but natural; but when the time allowed is unreasonably short and the penalty high the tendered prices are increased to cover the risk. Self-preservation is Nature's first law. If the time set apart for the job is short, then the manufacturer must obtain his material at once, and to do so offers higher prices, buying from stock, from mills, anywhere, in order to obtain speedy delivery. It is by no means uncommon for the constructional works to be asked to give (a) a tender, based upon incomplete drawings, within a week, and (b) to manufacture and erect a job in less time than that occupied by the design. The client presses for delivery, and the time lost in the design office has to be recovered elsewhere.

Incomplete Drawings. No words can fully convey the trouble entailed by this system of tendering on incomplete drawings. The unfortunate person who has to "take out" the weight, as a step towards ascertaining the cost of the structure, must allow for pieces and connections which are not shown; in other words, he has to guess (scientifically, no doubt, but nevertheless guess), and should he guess low, the firm probably secures the job. Generally, however, the uncertainty tends to keep the cost high.

On the other hand, the firm may knowingly cut their price, and this can only be done at the expense of weight and strength. Advantage is taken of the incomplete drawings to use slender scantlings, and the greater the temerity of the firm concerned the lighter the sections. That this is by no means exaggerated the following items, taken from competing tenders for the same building, show: roof trusses, span about 43 ft. at 11 ft. centres; purlins approximately at 6 ft. centres; length of building, 200 ft.

Firm A :

6 purlins @ $3\frac{1}{2}" \times 3\frac{1}{2}" \times \frac{3}{8}" \times 200'$ @ 8.45 lb./ft. = 10,140 lb.

4 purlins @ $3\frac{1}{2}" \times 3\frac{1}{2}" \times \frac{5}{16}" \times 200'$ @ 7.11 lb./ft. = 5,688 lb.

Total

15,828 lb.

Firm B :

10 purlins @ $2\frac{1}{2}" \times 2\frac{1}{2}" \times \frac{1}{4}" \times 200'$ @ 4.04 lb./ft. = 8,080 lb.

The purlins of firm B must presumably act as suspended cables, and not as beams!

Designs direct from Steelworks. It occasionally happens that the Consulting Engineer or Surveyor, etc., having only a slight knowledge of structures, sends inquiries direct to several constructional firms. There is enclosed a single line diagram of the

proposed bridge or structure, together with overall dimensions and the loads which the structure has to carry. He asks for drawings indicating the contractor's proposals, together with the price inclusive of erection. This method gives a still wider range for thinning down. Firms are competing on the vastly different levels set by their anxiety to get work and by the scientific knowledge of their staffs. One firm employs highly specialized men for designing, whereas another employs perhaps a single man, who has to turn his hand to designing, draughting, ordering of material, etc. The costs of these staffs are entirely different, and make themselves felt in the prices. It is a fact that the unskilled man nearly always designs light ; he cannot foresee all the forces or loads which may act, and generally thinks of standard cases of loading.

The question now naturally arises in the reader's mind, " But if the light structure stands up, why put up a heavier and therefore a more costly one ? " It is true that there is much to be said for this, but the reader would probably prefer to cross a bridge which is absolutely safe than one which is just on the margin of safety. A lightly designed structure is seldom rigid ; and rigidity in structures is a valuable asset, but involves, as a rule, increased weight. A structure may sway and deflect alarmingly and yet stand up. As a case in point, a Continentally manufactured tower crane was erected in this country which on testing carried its test load safely, but the deflections were so great that the working loads had to be considerably reduced from those originally intended. The factors of safety and good luck also help these " cut " structures, but in the long run they are not economical, and parts have to be renewed much sooner than would otherwise have been the case. After all, it is only common-sense to say that by dealing with a reliable firm the chances are in the client's favour.

The Engineer having received the various tenders and drawings, accepts, as a rule, the lowest or one of the lowest offers, but this does not necessarily mean the cheapest structure, since if it has been too lightly designed the upkeep will swallow up any saving in initial cost. This method of operation lends itself to some other objectionable practices, and a case is given in illustration. X. asked for and obtained prices and designs from various competing firms, and among these was a good set of dimensioned drawings from a firm of repute. This firm lost the contract, but were amazed to see their own design being erected. Inquiry proved that X., liking the design of the firm, had arranged with another firm to undertake it, with minor alterations, at a much lower figure.

Complete Drawings. The Consulting Engineer should make complete drawings, and these, together with a good specification,

form the only satisfactory basis of tendering. He has had time to weigh up the pros and the cons, and should have conveyed the job to paper exactly as he desires it to be before prices are asked. It is only fair to say that the latter is the practice of the majority of engineers, especially those belonging to the recognised institutions of engineers, but unfortunately any person, no matter what his lack of qualifications is, can call himself a "Consulting Engineer," and therein lies much of the trouble. The competing steel contractors, having been furnished with the drawings and specification, are all at "scratch" in the race; their yard equipment, their contracts with the rolling mills or with the stockyards for material, their erection schemes and transport of the work to the site, all furnish legitimate fields wherein money may be saved without danger to the client, because prices only become comparable when quoted for the same set of drawings, specification, and schedule of quantities.*

The more detailed the drawings submitted for prices the less are the chances of their being misread wilfully or unintentionally, and the less is the work entailed in estimating from them. A point which seems to be overlooked is, that if for one job eleven firms are asked to send in designs and tenders, then ten of them must spend money needlessly. It costs each of the competing firms, say, £50 to make out their estimate (on some occasions it has reached thousands for a single estimate and competitive design), making a useless and non-productive expenditure of £500. Who pays this? The only reply is, "The client himself," because he in turn is paying an enhanced price for some previous client's mistake. It follows that if every client who desired a price submitted a careful set of drawings, specifications and schedule of quantities, the cost of tendering would be less, and prices would tend to fall. Under the latter system only one office, that of the Engineer, does the work, and not eleven independent ones.

Although these practices are not common, the mere fact of their existence calls for a warning to be given to the student, and it is because no text-book draws attention to them that the writer has felt himself constrained to write rather fully upon them, knowing that he has the approval of many engineers in so doing.

Detailed Drawings are based upon the small scale design drawings, and should have been included amongst those sent out when asking for prices. These drawings show the different pieces of the structure to a large scale. Take, as an example, the case of a workshop. The design drawing of $\frac{1}{4}$ in. equal to 1 ft. scale would show the cross-

* During the last few years the competitive market has, apparently, not been entirely free. Estimates and tenders submitted by contractors for the same contracts have often exhibited a remarkable resemblance to each other.

section, outside elevation and plan, and any additional views necessary. The roof truss on this drawing would have the sections written against each bar, together with gusset plate thicknesses. The rivets would not be shown as such, but would be indicated at the end of each bar as 3 r., 2 r., etc., the diameter of the rivet hole being specified in a prominent footnote. For an example, see Fig. 131, Vol. II.

The draughtsman, under the superintendence of the designer, redraws the truss on one sheet (or sometimes half the truss when it is symmetrical).

The scale of this detail drawing is usually $\frac{3}{4}$ in. or 1 in. equal to 1 ft., while special details may be enlarged to $1\frac{1}{2}$ in. equal to 1 ft. This sheet would then give the main or leading dimensions of the truss, such as height or rise, and span, etc. This fixes the outline of the truss, and the smaller bars, forming the web or interior portion of the truss, then automatically fall into position. Because these subsidiary bars are, with regard to length, functions of the leading dimensions, some drawings do not quote their length. The exact fixing of length is thereupon left to the template-maker to measure from his full-size lay-out.

The lengths of these bars can be scaled from the drawings fairly accurately, and should the draughtsman want to "cover" himself, he can always add after the dimension the contraction "abt." for "about." To a draughtsman familiar with "logs" there is no great difficulty in calculating their lengths. The point is that by noting the lengths of the secondary bars on the drawing the draughtsman facilitates the ordering and sorting out of material. The system of wood template making has encouraged this habit of not stating the lengths of subsidiary bars.

The drawing shows every hole and rivet, and fixes their position by giving dimensions to some important point in the structure, such as a panel point, etc. Holes which are to go open to the site are ink-filled on the drawing to differentiate them from holes which are riveted up in the shop.

The scantling and size of every section is mentioned alongside the bar concerned, and all printing and dimension lines are kept clear of the actual drawing itself. It is better to err on the full side with information, but repeating obvious information results in a congested drawing which is difficult to read.

Usually this drawing is made on a cheap cartridge paper or a thin detail paper in pencil line. It is next forwarded to the tracing office, where girls trace the drawing on tracing cloth. In some offices the drawing is made straightway upon the tracing cloth, the dull side of the cloth being used, as it takes a pencil line. The lines

about which there is no dubiety are inked in at once, the pencil work keeping pace until the whole tracing is completed.

Ordering Material. The foregoing detailed drawings may be made previous to entry of the job to the constructional works or after entry. In any case a job number is assigned to it, and all subsequent time and material is charged against this job number.

Material can now be ordered either from the mills or from the steel merchants' stockyards with every degree of accuracy. The sections are grouped under their respective scantlings, lengths of small pieces summed and ordered as long bars.

Joist scrap, *i.e.*, cuttings and odd lengths, has little useful value in constructional works, and is sent back at scrap value to the smelters. Channel scrap is perhaps slightly more useful, but in any case these bars should be ordered from the mills as near the exact length as is possible. It is advisable to arrange the order list so that there is a large number of bars of the same length instead of every bar being of a different length, and when ordering from the mills the 2 in. cutting margin should be remembered.

Angles may be accepted, say from a steel merchant's stockyard, from several inches to several feet longer than is required for the job on hand. This depends greatly upon the section. The extra lengths are put into the stock of the constructional works and are drawn upon to make cleats, small stiffeners or short bar members of roof trusses. The important point is that odd lengths of angles have a use in the constructional works, and are not merely scrap steel.

Plates and flats give valuable scrap, their cuttings and trimmings being used for small gusset plates and washers. Small plates are cut out of long plates, and when ordered are so arranged that the small pieces fit into each other neatly, without much intervening waste space. Care is taken that this entails no re-entrant cuts. If plates are to be planed, the $\frac{1}{8}$ in. or $\frac{3}{16}$ in. allowance for planing each edge should be added to the ordered width.

The order sheets are typed through carbon papers or written by hand so that several copies are obtained. An extra column is generally given on these sheets, wherein are briefly stated the sizes into which the ordered lengths are to be subdivided on their arrival at the yard, and also the position they occupy in the structure. Obviously this is a valuable help to the yard.

In addition to the copies required by the order clerk and the drawing office for reference, copies of the material list, together with blue prints of the working drawings, are sent to the template loft and the yard.

REFERENCES

COLEMAN and FLOOD. *Civil Engineering Specifications and Quantities* (Longmans, Green & Co.).

British Standards Institution. *Engineering Drawing Practice*, No. 308 · 1953.

CHAPTER III

THE TEMPLATE LOFT AND THE WORKS

THE TEMPLATE LOFT

TEMPLATE-MAKING is the next stage in the fabrication of a structure.

The roof truss, which has been already mentioned, is now drawn full size upon the template (or templet) floor from a base and a centre line. Long lengths are measured with a steel tape and are struck with a chalked cord. The actual length of every bar is then obtained from this drawing.

The templates may be either (1) the bars which are to be used in the structure or (2) made from some light and easily worked material.

Method 1 calls for a highly skilled plater, who does all his own measuring and setting out of the work. The templates are the actual sections after they have been straightened. One bar is therefore finished, and other bars similar to it are marked from it. The plater is thus always in touch with his job after it leaves the drawing office, and becomes more conversant with it as it approaches completion. Further, since it is the bars of the structure which are used for templates, any irregularity due to sectional growth is allowed for at once.

This practice rules in America, but has very few followers in Great Britain.

Method 2 employs wooden strips which are made to the full size lengths obtained directly from the floor. Holes are drilled in them where holes have to be drilled in the finished bar. After completion the wooden templates are despatched to the yard. Instructions are written on them in heavy pencil or paint to direct the "marking off" of the actual bars. This is the practice in Great Britain. It has one drawback in that it reacts disadvantageously upon the drawing office, which is inclined to "leave it to the loft."

It is argued against this system that, in addition to the plater, the template-maker has to "worry out the drawings" also, to which the reply is that wood templates considerably simplify matters for the plater and save raw material from being wasted.

Briefly, then, method 1 requires more painstaking work from the D.O. than is common in this country; method 2 employs the

extra man, the template-maker, with a saving in time of the D.O. It is a moot point as to which system is the better, but if practice counts for anything, it may be taken that the second method, probably due to the quality of labour obtainable, is cheaper; otherwise it would not be perpetuated.

The Templates are generally strips of wood $\frac{3}{8}$ in. to $\frac{1}{2}$ in. thick and of varying width up to 6 in. or even wider. Several strips may be fastened together by straps to give extra width or a skeleton frame when necessary. It will be understood that these strips represent the working surfaces of the bars. In the case of an angle one strip may represent one or both legs, since it functions as a transfer. The wood for good templates is pine; for cheap templates, white deal.

Thin gauge iron or steel makes excellent templates for accurate work. They are indestructible, and are affected by temperature in exactly the same manner as the parent members for which they are intended. Wood, on the other hand, is always seasoning, and climatic conditions call for care if accurate workmanship is to be maintained. Three-ply wood is good, and is more trustworthy than single ply. Cardboard, millboard and brown paper are useful, but special template paper is now universally employed.

The loft floor is long and wide, with the template-makers' benches set round the wall. The cost of ground and the necessity for good light has caused the floors of most template shops to be at least one flight up. This has one advantage, as it keeps traffic off the floor, which is sometimes specially blackened once a year.

The remarks made about congested drawings are equally applicable to templates. The template-maker, to save wood and time, often makes a template strip do duty for an ordinary repeat part of the structure and also for special bars. The special bar may be an ordinary bar with additional holes and connections. Again, the bars may be so many left-handed and so many right-handed. It follows, therefore, that if the template is made to do duty for the lot it has a large list of instructions written on it, with the corresponding holes encircled in various colours to indicate which of them shall be taken. Some templates are like jig-saw puzzles, and, instead of saving time, lose it, incurring, as they do, journeys to the loft for the solution. Old templates are used over again by sawing down and plugging the holes. Templates, unlike patterns, are not commonly stored against future use, since structures are not standardized like mechanical parts.

THE WORKS

Straightening of Material. While the work on the wood templates is nearing completion the raw material is being delivered from the

mills. Unequal rates of cooling at the mills and transportation have caused slight bends in bars and plates, and these must be cold-straightened.

Cold-rolling flattens the plates. The machine used for this purpose has a double set of horizontal rollers, the upper tier of rolls entering the space between the lower ones, just as one would stack pipes. The rolls are capable of adjustment, and plates can be made flat or curved for steel chimneys, tanks, etc.

Cold-straightening may be accomplished by hand-hammering in the case of light sections. The bar is placed upon two wedge-shaped supports with the convex face up, the bend being at mid-span. By striking the section at the bend the result is an obvious straightening action. Machines for straightening (or bending) are built on this principle, and are therefore glorified jim-crows which are used in tramway or railway work for rails.

In the case of machines the hammer is replaced by a ram driven by hydraulic or other power, and the action is similar to that of

hand-hammering. From the formula of $\frac{M}{I} = \frac{E}{R} = \frac{f}{y}$ the student

can easily arrive at the value of the extreme fibre stress set up by this mechanical bending or straightening. That this straightening, by bending, causes stresses well in excess of the elastic limit, is not generally realised. It is quite possible that a tension angle in a structure may already be carrying a high tensile stress at one portion of its length previous to its being built into position. If the section which has been mechanically straightened be edge-planed, it always exhibits a tendency to return to the original bend. As with a punched hole, the only method of wiping out the internal stress is to anneal the section, a process which is unthinkable.

Astragal tee bars are placed in special end grips and are pulled out by hydraulic power, as in a tensile testing machine. The writer on one occasion, using an ordinary steel tape, worked out the stress for a $1\frac{1}{2}'' \times 2'' \times \frac{1}{4}''$ tee astragal so straightened, and arrived at the figure of 26 tons per square inch. There was a slight slip in the grips, but the fact nevertheless remains that the bar was overstretched and therefore overstressed.

Cutting Machines. The bars are subdivided cold either by the revolving circular saw, by the cropping machine, by the shearing machine, by the guillotine, or by the cutting flame.

Circular Saw. The heavier sections are usually allotted to this machine. Much can be done with the circular blade, as this can be set to cut a predetermined distance in and then return, or it

can cut straight through. A $24" \times 7\frac{1}{2}" \times 95$ lb. R.S.J. can be cut through in fifteen to twenty minutes.

With the new mild steel disc saw an exceptionally high peripheral speed of 30,000 ft. per minute is obtained when running at 15,000 revolutions per minute. The blade can be raised and lowered at will, while the vertical plane of the blade may be inclined to the section at any desired angle and there locked. The sections can remain, therefore, always in the same position on the bed instead of being swept out into the shop, and so occupying valuable shop space when a bevel cut is required. The heaviest steel joist rolled, of 95 lb. per foot, can be cut through in about a minute and a half.

Flame Cutting is employed for all sections. Using oxy-acetylene, coal gas or hydrogen, a clean cut is obtained through any thickness of steel. The nozzle may be either hand controlled or mechanically guided along a path at any uniform speed (depending upon thickness) by track or template. The nozzle is adjustable to give either a straight or a bevel cut at any angle.

Shearing Machines have a heavy blade, about a foot long, inclined to the horizontal at an angle of 10 degrees or so. The cut is a matter of a few inches, while the number of strokes runs about twenty per minute. The plates get distorted, especially if long, due to the inclination of the descending blade when shearing its way through the plate. The resulting edge is ragged and uneven, and cuts the hands of the workers if they are not protected by rectangular pieces of leather with a slit through which the wrist passes.

The Guillotine is a larger machine than the shears, with a blade from 3 ft. to 10 ft. long and which has only a slight inclination to the horizontal. The resulting cut, if shorter than the blade, is straight and of good surface. While the plate is being adjusted in the machine the blade and the holding down beam remain stationary in their highest positions. On depressing the treadle or hand lever these descend, the beam leading, and on the cut being made return once more to their original positions. The weight of these machines runs from 4 to 30 tons.

The shearing action adversely affects the tenacity of the immediately adjoining fibres of the steel, but only for a short distance in. The punching of a hole has the same injurious effect. When sheared edges are used the rivet holes are kept further in from the edge than in the case of a rolled or planed edge. The extra allowance varies from $\frac{1}{8}$ in. to $\frac{1}{4}$ in. A slight fissure or crack often pre-determines the line of failure. With a sheared edge under tension there is always the possibility of one of the small cracks opening out and the plate tearing; under compression the cracks tend to close. Most specifications permit a sheared edge on the top or

compression edge of the web plate, but ask for a planed edge on the bottom or tensile edge.

Regarding Appearance. A flange plate with guillotined edges, especially when painted, is indistinguishable from a planed edge when viewed a few feet away.

Cropping Machines are, in effect, special shearing machines, with suitable openings for the various types of sections. For example, the angle cropper has a right-angled V-notch, which supports the legs against crushing when the blade is cutting through. Good clean-cut ends are obtainable under ordinary working conditions. In the newer type of machine all manner of bevels, or splays, can be given to the cut end of an angle or tee. This is in contradistinction to the older type, still in use, which can only give a straight cut at right angles to the bar's length.

Highly splayed ends should not be used for the following reason. The position of the rivet is definitely fixed by the rule that its centre shall never be nearer the edge than $1\frac{1}{2}d$, where d = diameter of hole. In a long splay the end of the bar is far removed from the rivet head, which tightly clenches the adjoining metal parts together and causes the end of the bar to bend up and leave the gusset plate. This gap encourages rust action, since a painter's brush cannot obtain entry. Splays are necessary in order to give clearance to some other adjacent bar (see the drawing of roof truss details, Vol. II.), otherwise a standard right angle, straight through cut is preferable.

Marking Off. With the bars, etc., cut to length the marking off is greatly simplified. The section, with the template clamped to its upper surface, is placed on a raised platform or bench. The holes in the wooden template guide the marking tool—a centre punch or a tube of the requisite diameter dipped in some cheap white paint or whitewash—so registering the position of the holes on the steel. The indent made by the centre punch is difficult to see, so that often both the punch mark and the vivid circle of colour are used conjointly.

The Punching Machine has an action almost similar to the hand punch used for holing paper for filing purposes. The punched circular bite is pushed through; the punch then ascends, and the bar is pushed along until the next mark comes under the now descending punch, and so on. Some specifications ordain that material of $\frac{5}{8}$ in. or $\frac{3}{4}$ in. thickness and over shall be drilled; others, again, prohibit the use of punched holes for all main members, but permit their use for secondary parts such as floor plating and lattice bars.

The punch destroys the tenacity of the annular ring of metal just surrounding the hole. Annealing restores the damaged metal to its

pristine strength, but the cost of such a process is prohibitive and out of all proportion to the amount of damaged metal. Two methods therefore present themselves. First, to punch a hole $\frac{1}{8}$ in. less in diameter than the finished hole and then reamer or clean out the hole to the proper diameter. The damaged metal is removed by this process. Secondly, to leave the damaged metal in position, but when calculating the net area of the section deduct from the gross area of the bar an imaginary hole the diameter of which is $\frac{1}{8}$ in. larger than the actual diameter of the finished punched hole. The latter method, which neglects the damaged metal still in place, rules in America and in some firms in this country.

The hole made by the punch is inclined to be slightly tapered in its length; further, since each plate must be punched separately and assembled, holes which take the same rivet cannot be in such accurate register as holes drilled from the solid with assembled plates in position.

Fig. 33 shows the assembled plates with punched holes which are not in exact register. (The holes have the taper exaggerated in the

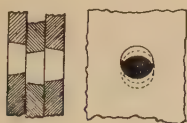


FIG. 33

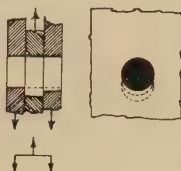


FIG. 34



FIG. 35

sketch.) The succeeding figure illustrates the holes after having been cleaned out or reamed for alignment, while Fig. 35 indicates how the finished rivet shank has a protuberance on it. The material for this excrescence can only be obtained from that metal which should have gone towards forming the final rivet head, which must perforce now be undersized.

Drilling. The material is removed by a high speed twist drill which leaves the metal surrounding the hole undamaged. By assembling the angles and plates which form the flange of a plate girder the drilling of each rivet hole from the solid and through the several added thicknesses, can be carried out at one operation. Where plates, etc., can be so bundled together, marking off is considerably reduced, and, above all, the holes are in exact register, especially if sufficient tacking holes and service bolts have been used. (Service bolts are rough bolts used and reused for temporary

fastenings, and are more trustworthy than clamps. The bolts are spread here and there over the job, and are taken out and replaced, etc., when too near the drill.) The plates and angles of flanges are then dismantled and all the burrs and turnings, which have been forced in between the plates, removed preparatory to riveting.

Where gusset plates are identical these can be bundled and the holes drilled through the set at one operation. Where these plates are $\frac{3}{8}$ in. thick and under, it is actually cheaper to have the holes drilled instead of punching them; this statement takes into account upkeep, initial cost, labour and marking off, etc. Where plates cannot be so bundled the punch is the cheaper instrument; on the other hand, the punched holes when assembled often require the extra process of reamering to bring them into alignment.

Where several radial drills are mounted in line they form a battery of drills. The radial arm swings round the pillar as centre, while the drill itself can travel horizontally along the cantilever arm. The drill can be set to any point and then locked preparatory to drilling. This drill does all the straightforward girder drilling. In addition there is, of course, the electric drill, with perhaps a magnetic grip, the pneumatic drill and the old-time ratchet brace. The last is often resorted to for site holes, especially when making connections to existing work.

The last three drills are small and portable, and have their own particular sphere of usefulness. Lack of space forbids a full description of these, and the student is referred to the makers' catalogues and the trade advertisements and articles in *The Engineer* and *Engineering*, etc.

Planing Machines are used for edge planing. Surface planing, as in bridge bearings, etc., is carried out on a surface-planing machine. Ordinary plates do not require to be surface-planed. In edge planing the tool travels, while the plate is at rest; the reverse is the case with surface-planing machines. The usual maximum length of plate which edge-planing machines take is generally in the vicinity of 30 ft. Large machines can be obtained which can plane 40 ft. at one cut, which takes one minute. Most of the smaller machines, although not all, have open ends in the cast iron end frames or pillars. This permits of the edge planing of plates which are longer than the machine. When such long plates have to be edge-planed it necessitates a shifting of the plate to complete the machining, a process which means the use of the overhead travelling crane and all the resetting of the plate for alignment. The logical result is that firms try to arrange joints so that no plate is longer than the tool travel of their longest machine; only one setting of the plate is therefore necessary.

Hand-riveting is used exclusively only in small structural yards or in an outside job where the small number of rivets does not warrant the use of a pneumatic outfit. The squad consists of four: two riveters, a holder-up, and the rivet-heater, who is generally a boy. The last-named heats the rivet until white-hot in an open top coke furnace with bellows underneath. This cylindrical furnace is easily transported, and stands about waist-high, with a diameter about half its height. The rivet is then thrown by the boy to the gang, who place it in the hole from the underside. The protruding shank is roughly "knobbed" down, and then finished off to shape with a snap die by the two riveters. The reaction to this hammering is provided on the underside by the holder-up and his dolly. The dolly is a long-handled hammer with a recessed head to fit round the original rivet head. A bent bar of iron hangs from the steelwork which is being riveted, and through the loop passes the shaft of the dolly. This distance, rivet to loop, is much less than loop to holder-up. The holder-up, by thrusting down his end of the handle, exerts a greatly intensified upward force on the rivet, and so counteracts the blows of the riveters.

Riveting, Pressure Machines. Specifications often state that "wherever possible the rivets shall be machine-driven, preferably by means of pressure machines of approved design." Two machines fulfilling the above condition are:—

Hydraulic Riveter. High pressure water is permitted to enter a small cylinder by opening the inlet valve controlled by a lever. The water forces the ram out, and the rivet, coming between the stationary and the movable dies, is closed in one stroke.

The "holder-up" is stationary, and forms an integral part of the frame. The water in the cylinder is then released by moving the lever in the opposite direction and so opening the exhaust valve. On the cylinder emptying the ram returns to the commencement of its stroke.

Compressed Air or Pneumatic Riveter. This machine is very similar to the hydraulic riveter in general outline, the power closing the rivet being obtained from compressed air in place of water. Messrs. De Bergue's 40 ton pressure machine for girder work requires only 4 cubic ft. of free air for each 1 in. diameter rivet. On a "straight run" 2,500 rivets, $\frac{3}{4}$ in. diameter, have been closed in a day by one machine, although a fair average is about 2,000.

Pneumatic Hammer, unlike the two previous machines, rains a series of blows, and from the noise it makes has earned the name of **Pom-pom**. It is virtually hand-riveting mechanized, since it is percussive and requires an external holder-up, pneumatic or otherwise.

The Pneumatic Chisel is mentioned here because it greatly

resembles the pneumatic hammer. In this machine the small stroke and rapidly acting piston, actuating the chisel, is also driven by compressed air.

Drifting. It is usually emphasized in specifications that bolts only shall be used for holding the plates together when riveting is in progress. The only drifting permitted is the *gentle* pulling of the plates into position. The drift is a tapered bar, and by threading it through a series of nearly coincident holes and judiciously levering it the plate can be gently eased into position. This particular clause owes its origin to the practice of forcibly making holes register. By driving the tapered drift into a set of holes not quite coincident the edges of the holes are rounded and the circular holes become elongated. The finished rivet cannot therefore be up to the calculated strength.

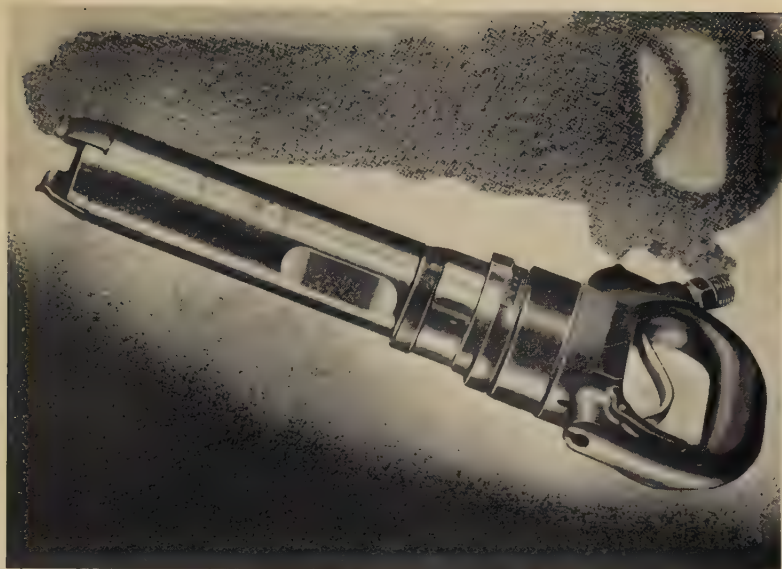
Painting. Mill scale can only be removed by weathering or by chemical means. It is becoming more customary to specify that work shall be despatched to site unpainted. By this means most of the scale has been weathered off, and the steel can be safely painted at site.

A designer takes pride in providing a section which has the exact area demanded by his calculations. But such skill is wasted if the structure is not adequately painted at erection and during its after life. In some British industrial cities unprotected mild steel corrodes at the alarming rate of 0.004 in. per year per exposed face. Thus after sixteen years a $\frac{1}{4}$ in. thick plate, unprotected on both faces, loses half its thickness.

Where poor maintenance of a steel structure is anticipated its useful life is sometimes expensively ensured by increasing the thickness of all its members by $\frac{1}{16}$ in.

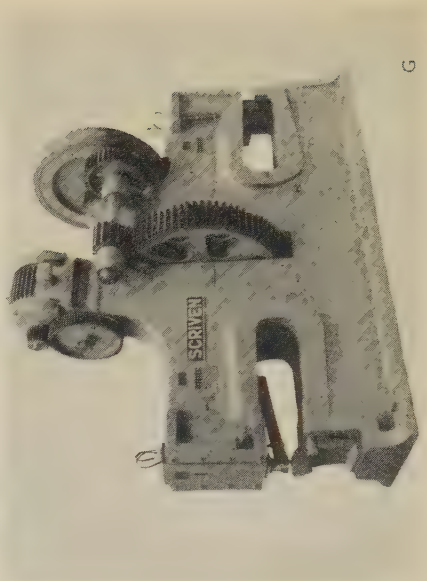
Copper bearing steels (as suitable as mild steel and only slightly more expensive) have a resistance to corrosion at least twice that of mild steel and their employment is fully justified if poor maintenance is expected. Copper bearing steels are now employed to give long life to steel sheet piling.

Erection. When the structure is to be sent abroad it is usual to specify that the whole, or part of it, should be erected in the yard of the constructional works. Thus, if the order is for several bridges all of the same span and not exceeding about 100 ft., one span at least is erected temporarily in the contractor's yard. The various elements composing the structure are then painted with a main position letter and a sub-position number, *e.g.*, G1 = Main Girder Diagonal No. 1; F3 = Floor Girder No. 3, etc. The general drawing is lettered to correspond with these markings. On occasion paints of different colours are used for coating the various parts of the



A

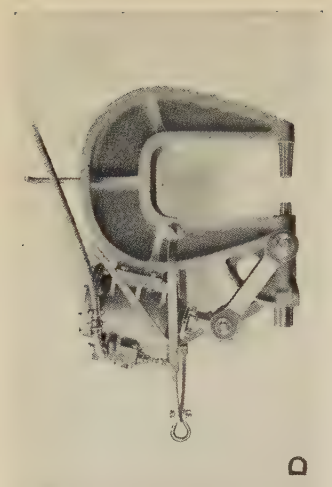
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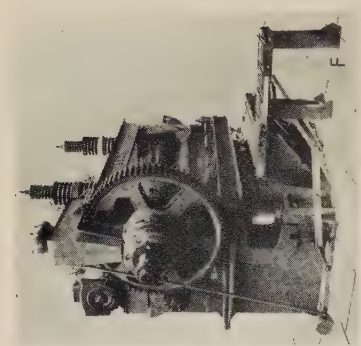
C



E



D



F



C

structure. In the case of a shop an end bay and an intermediate one are usually erected and covered with the sheeting. Here again every element and sheet is lettered to facilitate the final erection abroad. The original estimate includes an item for this temporary work for inspection purposes.

For home jobs no erection is carried out in the yard of the constructional works unless for special reasons. The system of marking the parts is maintained, however, as it proves itself to be of great assistance to the erectors at the site.

Shipment. On despatching the material abroad all sections are bundled together, since freight charges are based upon the overall bulk or the weight. There is one rate for volume and another for weight, and, needless to say, the higher of the two rates is charged. Thus similar gusset plates are piled together and bolted, while angles are nested into each other and wire-tied.

When the structure is to be erected at home the larger pieces are built up and riveted in the shops. The sizes of the pieces are limited by the transport facilities, the access to the site and the erecting equipment. Pieces which are exceedingly long or broad can only be sent by rail at a special rate after having been inspected and accepted by the officials of the railway company. Such pieces are only sent when the traffic is light.

ILLUSTRATIONS

A. *Angle and Tee Bar Cropping Machine.* (Scriven Machine Tools Ltd., Leeds.) Gives square or bevel cuts as shown by the specimens in the foreground.

B. *Pneumatic Riveting Hammer.* (Consolidated Pneumatic Tool Co. Ltd., London.) Piston diameter $1\frac{1}{16}$ in., stroke 6 in., 1,340 blows per minute., weight $21\frac{1}{2}$ lb. Overall length $17\frac{9}{16}$ in. Diameter of rivet (hot) is $\frac{7}{8}$ in.

C. *High-Speed Disc Saw.* (Scriven Machine Tools Ltd., Leeds.) Cuts joists up to 18 in. $\times$ 7 in. cold.

D. *Pneumatic Riveting Machine.* (De Bergue & Co. Ltd., Manchester.) A portable machine with a $24\frac{1}{2}$ in. $\times$ 14 in. gap for rivets 1 in. diameter.

E. *Plate Edge Planing Machine.* (Scriven Machine Tools Ltd., Leeds.) Electrically driven complete with hydraulic jacks and self-contained pump and accumulator. The tool is reversible to cut in both directions and a tilting motion is provided for planing bevelled edges. Cutting speeds between 25 and 60 ft. per min. Machines of somewhat similar type can be had to deal with plates up to 52 ft. long.

F. *Guillotine Shear.* (Scriven Machine Tools Ltd., Leeds.) For cutting plates 6 ft. 6 in. wide by 1 in. thick, gap 16 in. Heavier machines by the same makers can cut plates up to 10 ft. wide by 2 in. thick.

G. *Punching and Shearing Machine.* (Scriven Machine Tools Ltd., Leeds.) Will punch 2 in. dia. holes in $\frac{1}{2}$ in. thick plates, also shears $\frac{3}{4}$ in. thick plates. Gaps 36 in. Heavier machines can punch 2 in. dia. holes in and also shear 2 in. thick plates.

Note. The gear guards of the machines have been removed to show the details.

CHAPTER IV

FASTENINGS, PITCHES AND SIMPLE RIVETED JOINTS OF PLATES

Fastenings. Rivets and bolts are to steelwork what nails are to timber work. Rivets are the permanent fastenings; once completed, they cannot be undone without considerable labour, accompanied by the total destruction of the rivet. Bolts and nuts are the temporary fastenings used in steelwork, although they are often adopted as permanent fastenings. A paragraph on their use will be found at the end of this chapter.

By aid of these fasteners rolled sections of all types are stitched together, and the strength of the resulting structure depends just as much upon the rivets and bolts as upon the rolled sections. If a well-designed structure be purposely overloaded to produce failure, it should have every unit and element in it failing simultaneously.

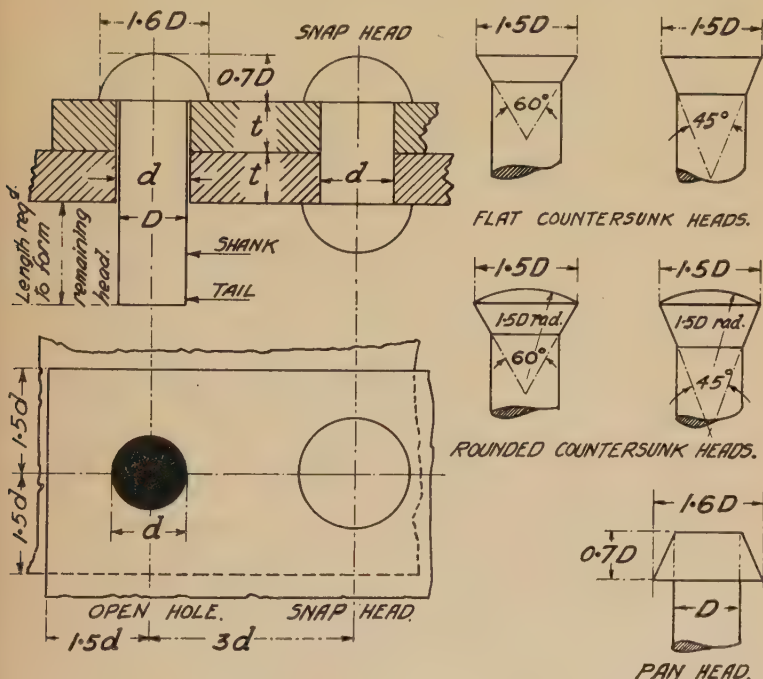
Rivet Material. The tensile breaking strength of rivet steel is from 25 to 30 tons per sq. in. with an elongation of not less than 26 per cent. on a gauge length of eight diameters. Wrought iron rivets were favoured by riveters and by some engineers for site work; at the site the facilities for riveting are never so good as in the shop, and wrought iron has the property of retaining its malleability after leaving the rivet forge or furnace longer than steel. Better formed rivet heads were consequently obtained when wrought iron was used. However, the use of wrought iron is not now countenanced, and the contractor has to make the riveting up to standard with the stronger material.

Rivet Heads vary in size according to the different trades which use rivets; and, not so many years ago, even in structural work, many forms of snap rivets were to be found. Within certain limits the question of the dimensions of the heads for rivets in shear and bearing would seem to be one more of appearance than of strength.

Fig. 36 illustrates the rivet dimensions adopted by the British Standards Institution in their *Standard Specification for Dimensions of Rivets* ($\frac{1}{2}$ in. to $1\frac{3}{4}$ in. diameter), and the reader is referred to this publication (No. 275—1941) for further information. This specification, however, does not apply to boiler rivets but represents a fair

average of the sizes which were used in British constructional work at the date of its first publication in 1927.

The conical-headed rivet is practically obsolete in constructional work, and in consequence does not appear in the schedule, while the



D is the nominal diameter of the rivet, i.e. the dia. of the cold rivet before heating.

d is the diameter of the rivet hole $= D + \frac{1}{16}$ ".

The rivet strengths are calculated upon the finished or hole diameter d .

BRITISH STANDARD RIVET HEADS.

(Reproduced by permission of the British Standards Institution.)

FIG. 36

most common head of all is the snap or cup head. Where construction demands that the rivet head should not project overmuch the rounded countersunk head is used, and where an absolutely flush or level surface is required—as in the sole plate of a roof truss shoe, or in the bearings of the bridge illustrated in a succeeding

chapter—one head is made flat countersunk. In the cases quoted the upper head is the snap and the lower one, in contact with the base plate, is the flat countersunk head, although occasions arise where it is necessary to have both heads countersunk, *e.g.*, in a bridge bearing where the upper rivet head fouls the bottom end of a tightly fitted stiffener.

Rivet Diameter. The hole in the plates is $\frac{1}{16}$ in. greater in diameter than the cold rivet shank previous to riveting. This clearance permits of the easy entry of the hot rivet shank into the hole. Just sufficient clearance should be given because the rivet, when being riveted under pressure, should squelch up and completely fill the hole. When the shank cools it contracts both in length and in diameter. The cold rivet heads therefore pull the plates together, while the shank cannot theoretically completely fill the hole. The larger the initial clearance the more difficult is the task of obtaining an ideal finished rivet which will completely fill the hole. It is usually specified that the initial clearance shall not be larger than $\frac{1}{16}$ in. nor less than $\frac{1}{32}$ in.

Owing to this clearance, ambiguity has long existed as to what a drawing specifying $\frac{3}{4}$ in. diameter rivets meant. Did it mean a $\frac{3}{4}$ in. diameter hole and the cold diameter of the shank previous to closing $\frac{1}{16}$ in., or did it mean $\frac{3}{4}$ in. diameter of cold shank with a $\frac{1}{16}$ in. diameter hole?

This ambiguity is easily removed by stating on the drawing that "The diameter of the rivet holes shall be $\frac{3}{4}$ in." This signifies an effective diameter of $\frac{3}{4}$ in. and a nominal diameter of $\frac{1}{16}$ in.

Nominal Diameter is that of the cold shank before driving.

Gross or Effective Diameter is the diameter of the hole or closed rivet. All rivet values are based upon this gross diameter. Preferably, the nominal diameter should be a multiple of $\frac{1}{8}$ in.

Rivet Grip and Ordering of Rivets. The grip of the rivet is the distance from the underside of one head to the underside of the remaining head of the completed rivet.

The length of the rivet as ordered from the makers is the grip plus the extra length required to form the second head.

There is always a certain amount of extra thickness, or "gather," where several plates are clenched by the same rivet, since plates do not lie mathematically flat against each other. In the case of long grip rivets, therefore, the ordered length is obtained by making an additional allowance which roughly increases with the number of plates. All constructional yards have their own lists giving the order length for a stated grip. The ordering of rivets only affects the works, and not the design.

Rivets closed by hand do not require the same length of shank as

machine-closed rivets, and are ordered from about $\frac{1}{8}$ in. to $\frac{1}{4}$ in. less in length. The machine-closed rivet therefore fills the hole in a more satisfactory manner. If the length, underside of head to tail or point of rivet shank, is too great, the rivet head when completed has a collar round its base; in appearance it may be likened to a washer fastened under the cup head. On the other hand, if the shank is too short an incomplete rivet head is formed. The rivet heads should bear tightly on the surface of the plate.

Rivet Line, also known as the scribe or back line and in America as the gage line, is the imaginary line along which rivets are placed. If the angle has equal legs, then the rivet lines are at equal distances from the heel, while if the legs be unequal, so also are the distances of the rivet lines from the heel, as in Fig. 37. Practice has assigned to each section rolled "standard" positions of rivet lines, which are conformed to wherever possible. The positions of these lines depend upon the flange or leg width, and are independent of the thicknesses of the bars. In the case of very thick angle bars little room is left for the closing of the rivet head by the adjoining leg of the angle (see p. 6). In these circumstances departure from the "standard" rivet line is sometimes necessary if the rivet diameter is to be maintained; alternatively a smaller diameter of rivet may be chosen in conjunction with the standard rivet line. The thickness of the bar, therefore, may affect the diameter of the rivet to be used. It is good practice always to dimension the position of the rivet line whether it is standard or not, as this eliminates errors both in the drawing office and in the works. In channel flanges the same rules are followed as for the legs of angles, while the flanges of joists are holed similarly to the tables of tee bars (see illustration to Table 8, Appendix).

Mnemonic for Angle and Channel Rivet Lines. To find the distance of the rivet line of an angle from the heel of the bar, add $\frac{1}{2}$ in. to the leg width and divide the total by 2. This rule holds good for angles of $2\frac{3}{4}$ in. leg and over. For angles of $2\frac{1}{2}$ in. leg and under substitute $\frac{1}{4}$ in. for the $\frac{1}{2}$ in. mentioned. In Fig. 37, since the bottom leg is 3 in., the rivet line dimension is $(3" + \frac{1}{2}") \div 2 = 1\frac{3}{4}"$; similarly for the 4 in. leg the dimension is $2\frac{1}{4}"$. For a $2\frac{1}{2}$ in. leg angle the dimension to the rivet line is $(2\frac{1}{2}" + \frac{1}{4}") \div 2 = 1\frac{3}{8}"$. These rules are also applicable to channels whose flanges have the same widths as the angle legs mentioned, and also to the stems of tees, the depth of the stem being equivalent to an angle leg.

Pitch. The pitch is the distance from centre to centre of adjoining rivets lying on the same rivet line. The pitch is kept

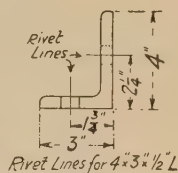


FIG. 37

constant over as large a length as is possible, and is not varied from rivet to rivet. It is better to have a large number of common pitches with an odd pitch cast to each end than to have all the pitches the same with each pitch containing, say, an odd $\frac{1}{16}$ in.

E.g., 1 pitch @ $3\frac{5}{8}$ " + 20 intermediate pitches of 3"
 + 1 end pitch @ $3\frac{1}{8}$ " = 5' 6 $\frac{1}{4}$ " total
 is better than 20 pitches @ $3\frac{5}{16}$ " = 5' 6 $\frac{1}{4}$ " total,
 as it causes the template-maker less trouble to mark off; besides,
 it is easier to check the job and thus keep down errors. The time

saved by the use of an easy pitch of 3 in. will more than pay for the two extra holes required.

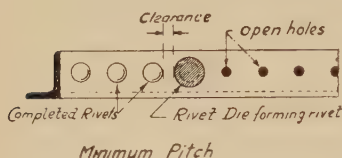


FIG. 38

The template-maker has often template strips marked and holed to the standard pitches of 3 in., 3 $\frac{1}{2}$ in. and 4 in. By setting off the two odd pitches first the inter-

mediate pitches are filled in at once by using the appropriate standard holed strip.

In roof trusses $2\frac{1}{2}$ in. and $2\frac{3}{4}$ in. are common pitches with $\frac{3}{4}$ in. diameter rivets. With plate web girders the most common and desirable pitches are 3 in., 3 $\frac{1}{2}$ in., 4 in., 4 $\frac{1}{2}$ in., 5 in. and 6 in. Pitches like $2\frac{3}{4}$ in., 3 $\frac{1}{4}$ in., 3 $\frac{3}{4}$ in., etc., are permissible, while odd sizes like $3\frac{1}{16}$ in., $3\frac{3}{16}$ in., etc., should be used as seldom as possible. The pitch multiplication table given in Table 17, Appendix, will be found useful, especially in plated work.

Minimum Pitch in structural work is specified to be three times the diameter of the finished rivet. If the rivets are spaced closer than this, very little clearance is left between the last-formed rivet head and the die forming the next rivet head. From Fig. 38 it will be seen that the absolute minimum pitch is half the diameter of the die plus half the diameter of the adjoining finished rivet head provided that the rivet head is in true position. Stating this as a formula, the absolute minimum pitch is roughly $2d + \frac{1}{2}$ ", where d = the diameter of the hole.

Where an angle has rivets through both legs care has to be exercised that proper clearance is given for riveting. In Fig. 39 the rivets are reeled; that is, the rivets in the vertical leg come midway between those in the horizontal leg. In Fig. 40 the rivets in the vertical and horizontal legs occur at the same cross-section. The 4" $\times$ 3" $\times$ $\frac{3}{8}$ " angle of Fig. 41 illustrates the problem of riveting which occurs in Fig. 40. Clearance is given to the die when closing rivet *B* if rivet *A* has been closed first. On the other hand, if rivet *B* be closed first, no clearance is given to close rivet *A*; the

position of the die for this case is shown in broken and slightly hatched line. The choice of the size of the angle and of the rivet diameter requires a little care on the beginner's part in order that the rivet heads do not foul.

Further, there is the weakening of the angle, due to rivet holes, to be considered. In the cross-section of the angle with reeled holes, Fig. 39, there is one rivet hole out, whereas in that of Figs. 40

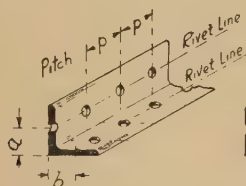


FIG. 39

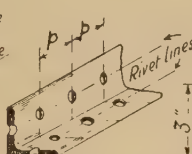


FIG. 40

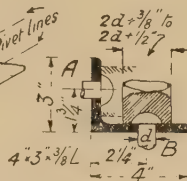


FIG. 41

and 41 there are two. It is, therefore, the practice that wherever possible rivet holes are reeled as in Fig. 39.

In Fig. 43 clearance is obtained for riveting by reeling the rivets, and the section is weakened by two rivet holes, not four, at any cross-section. It would be impossible in this case to have four rivets, all 1 in. in diameter, placed on the same cross-sectional line after the manner of Fig. 40.

Reeled Pitch, Alternate Pitch or Staggered Pitch occurs when an angle has a double rivet line on one or both legs, as in Figs. 42 and 43. The distance measured along one rivet line from the centre of a rivet on it to the centre of the adjoining rivet on the lower and parallel rivet line is known as the alternate pitch, reeled pitch or staggered pitch. This is in contradistinction to the word "pitch," which means from rivet to rivet on the same line.

Ambiguity may arise as to what a drawing means by the word "pitch" in the case of an angle having a double rivet line. The method of stating the pitch employed in Fig. 43a is to be preferred to that of Fig. 42a. There is not much to choose between the two methods, however, if the drawing is dimensioned to show what is actually meant.

When the angle has two 6 in. legs the riveting is reeled after the manner of Fig. 43. One rivet is near the angle root, while the other rivet, on the same cross-sectional line of the angle, is nearer the other toe. If the pitch is wide, the angle has only two rivet holes taken out of its gross cross-sectional area (see diagonal tearing, p. 76).

Where one leg is 6 in. and the other is $3\frac{1}{2}$ in. or so, the method of Fig. 42a and b is adopted. The $3\frac{1}{2}$ in. leg being approximately half

the width of the 6 in. leg, will be half as strong, and will therefore only require half the number of rivets. However, should it be

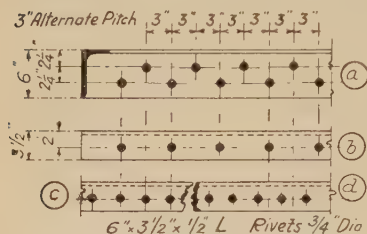


FIG. 42

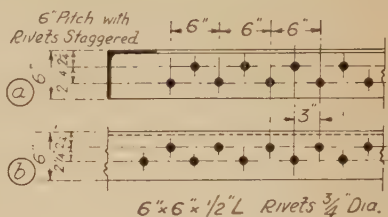


FIG. 43

desired to have a 3 in. pitch on the $3\frac{1}{2}$ in. leg, either system, as indicated by views *c* and *d*, may be used. Method *c* cannot be used if the rivets are large, because two rivets will occur on the same cross-sectional line close to the root of the angle, and will bring up the difficulty of clearance for riveting, as illustrated in Fig. 41. System *d* could be adopted where the rivet diameter is large. Method *d* is not necessarily more economical in providing net cross-

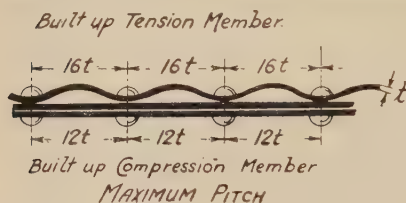


FIG. 44

sectional area than method *c*; see the rules on diagonal tearing discussed in the first article of Chapter V.

Maximum Pitch (Fig. 44).

If the rivets be widely spaced or pitched, the plates, especially if thin, are apt to gape apart. Specifications require

the maximum pitch to be not greater than "16*t*," where *t* = the thickness of the thinnest plate or section in the case of a built-up tension member. The same reasoning applies to a built-up compression member, but with the additional question of buckling. The various plates of a compression member, *e.g.*, a column, strut, or the top flange of an ordinary plate web girder, become little individual columns held to the neighbouring plates by the rivet heads. The rivet heads are under tension, and the wider the spacing the greater is the load on the rivet head, tending to break it away from the shank. Again, the further apart the rivets are the more flexible become the little columns, so that should one rivet head give way an individual column of twice the ordinary length occurs. This may finally lead to the entire failure of the built-up member. The maximum pitch in the case of a built-up compression member is limited to "12*t*" by some specifications, but others state

that "16t" may be used for tension and compression members alike.

Calculation in the case of a plate web girder may, for strength, only require a 5 in. pitch, but 12t, if $t = \frac{1}{4}$ in., would demand a 3 in. pitch. The extra rivets are added to stitch or tack the plates together.

The same reasoning applies to a reinforced concrete column where the links or rodding of the column are never spaced further apart than twelve times the diameter of the thinnest vertical bar.

Single Shear. Fig. 45 shows two plates being pulled apart, the upper plate, *A*, to the left, and the lower plate, *B*, to the right. If the rivet is weak, it will fail by shearing along the plane *xx*, the surface common to both plates. The area of metal sheared through is the circular cross-sectional area of the rivet shank, *i.e.*, an area equal to the plan area of the hole. If F_s be the safe working single shear stress in tons per square inch, then one rivet will safely carry a load

$$\text{of } F_s \times \text{area of hole} = F_s \times \frac{\pi d^2}{4} \text{ tons} = 0.7854 d^2 F_s, \text{ where } d =$$

diameter of the hole in inches, and n times this if there are n rivets.

Double Shear can only occur when there are at least three plates or members riveted together with a common rivet. Referring to Fig. 46, the plate *B* will travel to the right, on the failure of the rivet, with a piece of the rivet shank embedded in the hole. The rivet has thus been sheared at two sections: *xx* and *yy*. Since there are two single shears brought into action simultaneously, this is known as double shear. Most specifications assume double shear to be twice as strong as single shear.

If F_{ds} be the safe working double shear stress in tons per square inch, then the load which one rivet will safely carry is $F_{ds} \times \text{area of}$

$$\text{finished shank or hole} = F_{ds} \times \frac{\pi d^2}{4} \text{ tons} = 0.7854 d^2 F_{ds}. \text{ Alterna-}$$

tively, since there are two single shears, then the safe load per rivet in tons $= 2 F_s \times 0.7854 d^2 = F_{ds} \times 0.7854 d^2$, where d is the diameter of the hole in inches.

Crushing or Bearing of a Rivet or Plate. If a cylindrical bar of steel be laid lengthwise on some plastic material (like putty), it will gradually sink into the mass. Place the rod vertically with one end resting on the plastic substance, and its rate and depth of penetration will be greater in this case than in the previous one.

The weight of the rod is constant, so that penetration must be a function of the areas in contact. It has been found experimentally that the bearing resistance varies directly as the *projected area of contact* at right angles to the direction of motion. In case 1 this

was a rectangle equal to the length of rod by the diameter of rod ; in case 2 the area was a circle equal to the cross-sectional area of the rod.

Fig. 50 indicates the rivet crushing the plate ; similarly if the

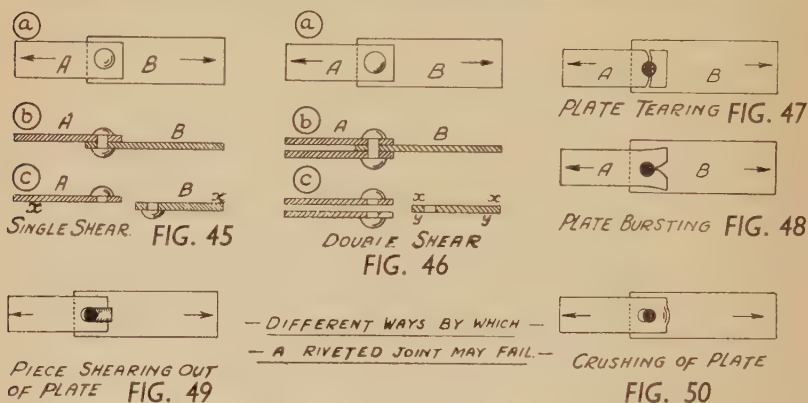
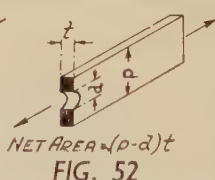
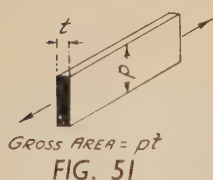


plate were the harder material it would crush the rivet. Crushing and shearing of a rivet are thus quite different.

If F_b = permissible bearing stress in tons per square inch, then the value of a rivet of diameter d bearing on a plate of thickness $t = F_b \times d \times t$ tons.

The bearing value of a rivet can therefore be raised either by thickening the plate or choosing a larger diameter of rivet or by increasing both.

When calculating the bearing value of a rivet with a countersunk



head only one-half the depth of the c's'k portion is to be considered as effective ; the diameter to calculate on is the ordinary rivet diameter.

Failure of Riveted Joints.

(1.) Plate tearing across at the weakest section, i.e., where there is least metal, as at a rivet hole (Fig. 47).

In Fig. 51 the whole cross-sectional area can be utilized to carry a tensile force or pull. If F_t = permissible tensile stress per square inch then the permissible load it can carry previous to holing is $F_t \times p \times t$.

The safe tensile load after holing (Fig. 52) = $F_t \times (p - d) \times t$, since only the **Net Area** can carry a load. To approximate to the net area of a plate with a c's'k hole in it, specifications assume the hole to be $\frac{1}{8}$ in. larger in diameter than the actual diameter of the rivet.

2. The joint may give way, due to the single or double shearing of the rivet (Figs. 45 and 46).

3. Failure may occur through crushing, as previously explained (Fig. 50).

4. The plate may rupture or burst in front of the rivet, as in Fig. 48.

5. Finally, the end piece of plate may be sheared out of the plate (Fig. 49).

Edge Distance and Margin. It is obvious that the nearer the hole is to the edge of the plate the greater is the chance of failure by either 4 or 5, as explained above. Experience has shown that if the margin between the edge of hole and edge of plate is at least equal to the diameter of the rivet, failure does not occur. Or, as shown in Fig. 36, the edge distance from the centre of the hole to the edge of plate is equal to $1\frac{1}{2}d$ (as a minimum), where d = the diameter of the hole.

In the case of sheared edges, where the metal may have suffered injury due to being sheared, the edge distance is increased to $1\frac{3}{4}d$.

Thus, if the rule for edge distance is followed, failure can only occur by one of the ways mentioned in 1, 2 or 3 or any combination of these three.

Efficiency of a Joint is the ratio of the least strength of a joint—i.e., by 1, 2 or 3—to the strength of the original plate previous to being holed. This fraction or ratio is usually multiplied by 100, and is thus expressed as a percentage.

Long Grip Rivets. When the grip of the rivet becomes long the rivet is subjected to bending in addition to bearing and shearing stresses. To allow for this extra stress on the rivets, most all specifications state that rivets carrying calculated stress, and whose grip exceeds 4 diameters in length, shall be increased in number by 1 per cent. for each additional $\frac{1}{16}$ in. grip. The grip has, therefore, an important bearing upon the fixing of the rivet diameter.

Rivets through Packings. The beam action of the rivet and its accompanying bending action are perhaps more apparent in this case. See Fig. 53. When the packings exceed $\frac{3}{8}$ in. in thickness the minimum number of rivets given should be the calculated number increased by 20 per cent. The additional rivets ought to be placed outside one of the connected members after the fashion of Fig. 54. Some of the load of the two lower main bars is picked up by the bottom rivets and transferred into the lengthened packer, and ultimately into the two outside upper plates. This lengthened packing is, in effect, equivalent to the swelling up of the two centre plates into a large "monolithic" head held between the two outside top plates. The load in the two main lower plates "perco-

lates " through to the packings, and is distributed approximately evenly throughout this head (Fig. 55).

The rivets on cooling contract and grip the plates tightly together. Due to this, a frictional resistance is set up between the various plate surfaces, resisting motion when the ends of the plates are pulled, thus reducing shear.

That portion of the joint designated as the massive head is more monolithic than any other portion, since it has the greatest number

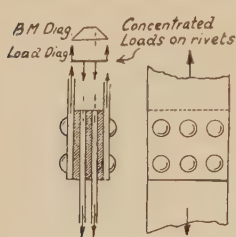


FIG. 53

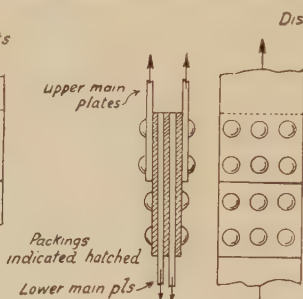


FIG. 54

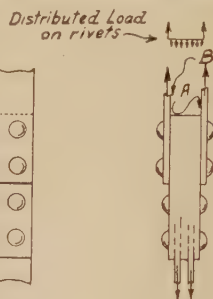


FIG. 55

of rivets through it. When frictional resistance fails it will do so on the inner faces, *A* and *B*, of the two outer main plates, and direct single shear will be set up at both these faces, accompanied, of course, by bearing. The bending action on the upper rivets, therefore, is reduced considerably. In fact, if the packers are carried sufficiently far down and attached by a corresponding number of extra rivets to the two mid-plates, the bending action on the upper rivets will be entirely eliminated. This assumes, naturally, that every rivet completely fills the hole through which it passes, and that its load = the total load $\div$ the number of rivets through the riveted member.

Working Stresses. It is found that the shear, crushing and tensile strengths bear a more or less definite relationship to each other, and so it is common usage to express the shear and bearing strengths in terms of the tensile strength.

From his study in "Strength of Materials" the student will be conversant with the fact that it is not the ultimate tensile strength (of 28 to 33 tons per square inch) which is used in designing, but a much lower value in order to be safe. The most common working tensile stress is 9 tons per square inch (*i.e.*, the factor of safety on the ultimate lies between 3 and $3\frac{2}{3}$). See the table of working stresses on p. xiv.

Although this would apparently mean that the actually occurring

tensile stress and load could be increased threefold before complete destruction ensued, the tensile member would be of no practical use long before this intensity of loading could be reached. The question of working stress and factors of safety are dealt with more fully in the latter half of Chapter IX.

F_t = permissible tensile stress in tons per square inch on net area.

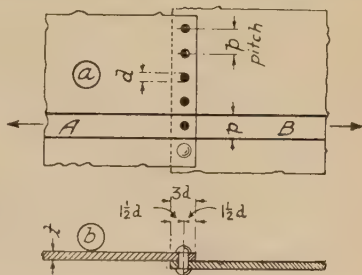
F_b = permissible bearing or crushing stress in tons per square inch } $= \frac{4}{3} F_t$.

F_s = permissible single shear stress in tons per square inch $= \frac{2}{3} F_t$.

F_{ds} = permissible double shear stress in tons per square inch of finished rivet shank cross-section $= 2F_s$ } $= \frac{4}{3} F_t$.

SINGLE-RIVETED LAP JOINT

When working out efficiencies it is immaterial from a strength point of view whether a pitch width or a 20 ft. width of plate be considered; what applies to a pitch width applies to any multiple of the pitch. Consider Fig. 56, where the plates have a single row of rivets and are lapped, by the foregoing rules, $1\frac{1}{2}d$ on each side of the rivet centre line, viz. $3d$ total. Take that portion AB of the plates shown in heavy line, which is a pitch in width.



SINGLE RIVETED LAP JOINT
FIG. 56

$$\begin{aligned} \text{Plate Efficiency} &= \frac{\text{tensile value of plate after holing}}{\text{tensile value of plate previous to holing}} \\ &= \frac{(p-d)t \times F_t}{ptF_t} = \frac{p-d}{p} \dots \dots \dots 1 \end{aligned}$$

$$\begin{aligned} \text{Shear Efficiency} &= \frac{\text{shear value of riveting}}{\text{tensile value of plate previous to holing}} \\ &\text{and, since } F_s = \frac{2}{3} F_t, \\ &= \frac{0.7854d^2 \times F_s}{ptF_t} = \frac{0.524d^2}{pt} \dots \dots \dots 2 \end{aligned}$$

$$\begin{aligned} \text{Bearing Efficiency} &= \frac{\text{bearing value of riveting}}{\text{tensile value of plate previous to holing}} \\ &\text{and, since } F_b = \frac{4}{3} F_t, \\ &= \frac{dtF_b}{ptF_t} = \frac{1.333d}{p} \dots \dots \dots 3 \end{aligned}$$

for this type of joint can never rise above 44.4 per cent. Thus with $3d$ pitch the efficiencies of **1** and **3** are $66\frac{2}{3}$ per cent. and 44.4 per cent., and with a pitch of $4d$ they are respectively 75 per cent. and 33.3 per cent. The economical pitch of $2\frac{1}{2}d$ is too close for ordinary work.

Rules for Rivet Diameters. From the above it will be seen that the theoretical size of rivet cannot be used because of practical considerations. Where a rolled section, such as an angle, is to be riveted to a plate, the position of the rivet line ordains the maximum size of rivet possible, because of the necessary clearance required by the die next the adjoining leg. Thus a $\frac{3}{4}$ -in. diameter rivet is the largest which can be used in a $2\frac{1}{2}$ -in. angle leg.

The rivet grip should not be over-long and the rule for grip of 4 diameters should be borne in mind. If the grip is long, then the diameter should be proportionately larger.

Again, in a girder or roof truss, sometimes even throughout a whole job if small, the rivet diameter is kept constant to save changing of drills, and also to save different sizes of rivets at site. As a rough guide the following approximate classification is given.

About $\frac{1}{2}$ in. dia.	Reserved for very light work, such as tanks, light steel doors, etc.
„ $\frac{5}{8}$ „	Light roof trusses of small span, say up to 25 ft.
„ $\frac{3}{4}$ „	Roof trusses, light lattice and plate web girders.
„ $\frac{7}{8}$ „	Large-span roof trusses (100 ft., say) and medium to heavy lattice and plate girders.
„ 1 „	For heavy work, <i>e.g.</i> , 80-ft. span double-track railway bridge.
„ $1\frac{1}{8}$ „	Exceptionally heavy work, and is employed only in the special parts requiring the heavy rivets, and if possible not throughout the whole structure.
„ $1\frac{1}{4}$ „	Have been used in constructional steelwork but seldom, and only in such structures as Hell Gate Arch Bridge, etc.

No one can be dogmatic on the classification of rivet diameters, as every job must be considered, more or less, on its own merits.

The student will, however, be safe in limiting himself to the choice of rivet diameters of approximately $\frac{3}{4}$ in. to $\frac{7}{8}$ in. When he has to design heavy structures he will be awarded the job because of the extent of his knowledge, and will settle the size of the rivet in most cases apparently without thought.

The following are two well-known rules for plate joints, but are not of much use in constructional steelwork.

Unwin. $d = 1.2\sqrt{t}$ to $1.4\sqrt{t}$, where t = plate thickness and d = the rivet diameter.

Another rule is, " d shall not be less than t ."

SINGLE-RIVETED BUTT STRAP JOINT, FIG. 57

The two original plates are made to butt, while the covers are placed over and under the joint as in (c), or a thicker cover over one face of the joint as in (b). General practice gives the covers the thicknesses shown, so that if failure occurs by tearing of plate, it will be the main plate which tears. With structural work, however, the cover area is not given on quite such a lavish scale. It is usually stated that when a joint is symmetrically covered—Fig. 57 (c)—the minimum area of the covers shall be 5 per cent. in excess of that of the parent section, and 10 per cent. in excess when the joint is unsymmetrically covered—Fig. 57 (b)—as against the total of 25 per cent. shown on the diagrams.

The width of the covers must be at least $6d$ in order to give an edge distance of $1\frac{1}{2}d$ to all four edges of plates and covers.

In (b) the rivet is in single shear with minimum bearing on the main plate.

In (c) the rivet is in double shear with minimum bearing on the main plate, the two covers presenting a greater bearing thickness than the main plate. (Fig. 46 illustrates this point as well as double shear.)

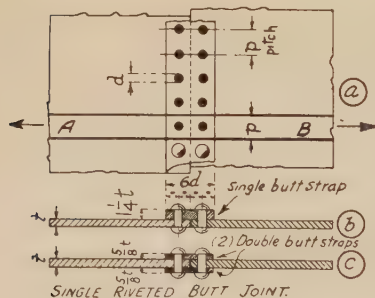


FIG. 57

Numerical Example. Fig. 57 (a) and (c), i.e., double covers. Main plate $\frac{3}{8}$ in. and minimum pitch limited to $3d$. To design the joint for maximum efficiency:—

$$\text{Plate Efficiency : } \frac{(p-d)tF_t}{ptF_t} = \frac{p-d}{p} \quad \dots \dots \dots 9$$

$$\text{Shear Efficiency : } \frac{0.7854d^2F_{ds}}{ptF_t} = \frac{1.047d^2}{pt}, \text{ since } F_{ds} = \frac{4}{3}F_t \quad \dots \dots 10$$

$$\text{Bearing Efficiency : } \frac{dtF_b}{ptF_t} = \frac{1.33d}{p}, \text{ since } F_b = \frac{4}{3}F_t \quad \dots \dots 11$$

For all-round maximum efficiency $9 = 10 = 11$.

$$9 = 10 \quad p - d = 1.047d^2 \div t, \quad \text{whence } p = (1.047d^2 \div t) + d. \quad 11a$$

$$9 = 11 \quad p - d = 1.33d, \text{ whence } p = 2.33d. \quad \dots \dots 11b$$

$$10 = 11 \quad \frac{1.047d^2}{t} = 1.33d \text{ and } d = 1.2732t. \quad \dots \quad 11c$$

The rivet diameter is fixed by 11c as being $1.2732 \times \frac{3}{8} = 0.48''$; adopt $\frac{1}{2}$ in. The minimum pitch of $3d$ supersedes equation 11b, so that the pitch used must be $1\frac{1}{2}$ in. The respective efficiencies of 9, 10 and 11 are now $66\frac{2}{3}$ per cent., 46.5 per cent. and 44.4 per cent.

Maximum Efficiency of Single Riveted Joints. So long as the minimum pitch is limited to 3 diameters, the joint efficiency can never exceed 44.4 per cent. by equation 11.

Diagonal Pitch and Distance apart of Rivet Lines. The distance between rivet centres, whether on the same rivet line or not, is limited to a minimum of $3d$ for riveting. The absolute minimum may be slightly less, as explained in the text referring to Fig. 38, but is only used in exceptional circumstances.

In Fig. 58c the three rivets *C*, *D* and *E* form the corners of an equilateral triangle, where each side is of the minimum length $3d = \text{pitch } p$. The distance *e* between the rivet lines is thus

limited to a minimum of $\frac{\sqrt{3}}{2} p = 0.866 p$, so that if *e* is made $\frac{7}{8} p$

($= 0.875 p$), the diagonal distance *CD* is at the least $3d$.

The diagonal distance is never given on a drawing, as it affords no help to the template maker, who is interested only in the "rectangular co-ordinates" of the rivet centres, *viz.*, the edge distances, distance between rivet lines, and the pitch. If the designer makes $e = \frac{7}{8} p$, or (as is usual in structural work) $= p$, he knows that the diagonal distance is more than $3d$ and that easy riveting is possible.

In Figs. 42 and 43 the pitch is 6 in. in the 6-in. legs, and therefore the distance *e* apart of the rivet lines can, from the point of view of riveting, be closer than $\frac{7}{8}$ of $3d$. This at once follows, because the 6 in. pitch is not the minimum pitch of $3d$. If the rivets had been 1 in. diameter the distance apart of the rivet lines, although only $2\frac{1}{4}$ in., would be still satisfactory.

Considering one pitch width of plate, shown shaded in Fig. 58c, and forgetting the existence of the plate on either side of it, there is the possibility of the strip tearing—

- | | |
|---|---------------------------------------|
| (a) along lines <i>EK</i> or <i>FH</i> ; net length = | $(p - d) = 2d$. |
| (b) " " <i>EGH</i> " = | $4\frac{1}{2}d - \frac{3}{2}d = 3d$. |
| (c) " " <i>EGK</i> " = | $6d - \frac{3}{2}d = 4d$. |

The B.S.S. No. 153 states that if there is the possibility of a plate tearing diagonally, then only $\frac{4}{5}$ of the net diagonal length shall be considered as being effective. Taking this into consideration, (b)

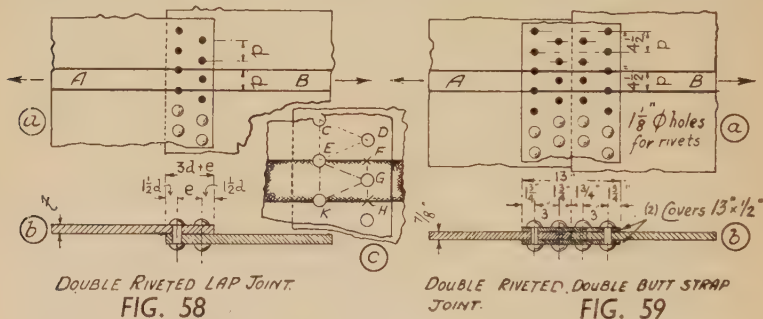
and (c) are still much longer than (a), and, therefore, in the event of failure by tearing it will be straight across from *E* to *K*, or *F* to *H*, as in our previous calculations.

The question of zigzag tearing is dealt with more fully in the opening article of Chapter V.

DOUBLE-RIVETED LAP JOINT

From Fig. 58 it will be seen that a pitch width of plate contains two rivets, *viz.*, two halves and one complete rivet. The shear and bearing equations will therefore have the coefficient **2** in the numerator.

Numerical Example. Design a double-riveted lap joint for two $\frac{3}{4}$ -in. thick plates so that the joint may be as strong as possible ;



minimum pitch is limited to $3d$, and the diameter of the rivet must not exceed 1 in.

$$\text{Plate Efficiency: } \frac{(p-d)tF_t}{ptF_t} = \frac{p-d}{p} \quad \dots \quad 12$$

Shear Efficiency :

$$\frac{2 \times 0.7854d^2 \times F_s}{ptF_t} = \frac{1.0472d^2}{pt}, \text{ since } F_s = \frac{2}{3}F_t \quad \dots \quad 13$$

$$\text{Bearing Efficiency: } \frac{2dtF_b}{ptF_t} = \frac{8d}{3p}, \text{ since } F_b = \frac{4}{3}F_t \quad \dots \quad 14$$

If failure occurs when the ideal joint is tested to destruction, then **12**, **13** and **14** should fail simultaneously, *i.e.*, **12** = **13** = **14**.

$$\mathbf{12 = 13} \quad p-d = 1.0472d^2 \div t, \text{ whence } p = 1.3963d^2 + d. \quad \mathbf{14a}$$

$$\mathbf{12 = 14} \quad p-d = 2.67d, \text{ whence } p = 3.67d \quad \dots \quad \mathbf{14b}$$

$$\mathbf{13 = 14} \quad 1.396d^2 = 2.67d, \text{ whence } d = 1.9'' \quad \dots \quad \mathbf{14c}$$

Equation **14c** gives an impossible diameter of rivet, so use the usual maximum of 1 in. The substitution of this value in **14a** and

14b gives a pitch of 2.4 in. and 3.67 in. respectively; the former value is not permissible owing to the $3d$ rule.

The respective efficiencies for 12, 13 and 14 are, with a 1-in. diameter rivet at 3.67-in. pitch, 72.7, 38.0 and 72.7 per cent.

The above shear efficiency is low and, obviously, can be increased by using the minimum pitch of $3d$. Adopting 1 in. diameter rivets at 3 in. pitch, the respective efficiencies are now $66\frac{2}{3}$ per cent., 46.5 per cent. and 88.9 per cent. By closing up the pitch from 3.67 to 3 in. the joint efficiency has risen from 38.0 per cent. up to 46.5 per cent.

Maximum Efficiency of a Double-riveted Lap Joint cannot be larger than 72.7 per cent., no matter what the diameter of the rivet and plate thickness may be. This is observable from equations 13 and 14, which give $t = 0.393d$; using this value and a pitch of $3.67d$, the efficiencies will run out at 72.7 per cent.

DOUBLE-RIVETED, DOUBLE-BUTT STRAP JOINT, FIG. 59.

$$\text{Plate Efficiency:} \quad \frac{(p-d)tF_t}{ptF_t} = \frac{p-d}{p} \quad . \quad . \quad . \quad . \quad . \quad 15$$

$$\text{Shear Efficiency:} \quad \frac{2 \times 0.7854d^2F_{ds}}{ptF_t} = \frac{2.0944d^2}{pt} \quad . \quad . \quad . \quad . \quad . \quad 16$$

There are two rivets in double shear, and $F_{ds} = \frac{4}{3}F_t = F_b$.

$$\text{Bearing Efficiency:} \quad \frac{2 \times dtF_b}{ptF_t} = \frac{8d}{3p} \quad . \quad . \quad . \quad . \quad . \quad 17$$

Example. Design a double-riveted, double-butt strap joint for $\frac{7}{8}$ in. thick main plates; no restriction is placed upon the rivet diameter.

Repeat the procedure of the previous example.

$$15 = 16 \quad p - d = 2.0944d^2 \div \frac{7}{8}, \quad \text{whence } p = 2.394d^2 + d \quad 17a$$

$$15 = 17 \quad p - d = 8d \div 3 \quad \text{whence } p = 3.667d \quad . \quad . \quad 17b$$

$$16 = 17 \quad 2.0944d^2 \div \frac{7}{8} = 8d \div 3 \quad \text{whence } d = 1.114" \quad . \quad . \quad 17c$$

Equation 17c fixes the rivet diameter at $1\frac{1}{8}$ in., the nearest working size, and on substituting this value in 17a and 17b the pitch has the two values of 4.16 and 4.13 in. The working pitch is therefore $4\frac{1}{8}$ in.

Adopting $1\frac{1}{8}$ -in. diameter rivets at $4\frac{1}{8}$ -in. pitch and substituting these values in equations 15, 16 and 17 the respective efficiencies of 72.7 per cent., 73.4 per cent. and 72.7 per cent. are obtained.

Assuming that practice could make and use a rivet 1.114 in. in diameter, then the pitch obtained by substituting this value for d in 17a and 17b is, for both equations, 4.085 in.—again an impractical dimension. If this theoretical diameter and pitch of 1.114 in. and 4.085 in. be used in equations 15 to 17, then the efficiencies all work out at 72.7 per cent. exactly. It is because the theoretical diameter is departed from in favour of a practical size of rivet that the apparent anomaly in the matter of the pitch arises.

Covers should be $\frac{5}{8}$ of $\frac{7}{8}$ in. each = 0.54 in. For usual practice $\frac{1}{2}$ -in. plates would be adopted. Since the joint is symmetrical then, in accordance with specifications for structural work, only a 5 per cent. excess of cover area is necessary and therefore the $\frac{1}{2}$ -in. covers are quite satisfactory.

Maximum Efficiency of a Double-riveted Butt Strap Joint with double covers can never rise above 72.7 per cent., and this occurs with a pitch of $3\frac{2}{3}$ diameters. Should the pitch be greater than $3\frac{2}{3}$ dia. the value of the bearing efficiency falls below 72.7 per cent., while that of the plate efficiency rises above 72.7 per cent. If the pitch be

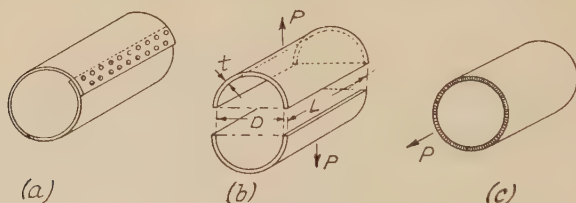


FIG. 60

less than $3\frac{2}{3}$ dia. the plate efficiency falls below 72.7 per cent., while the bearing efficiency rises above 72.7 per cent.

As a further example of a riveted joint the design of a riveted mild steel hydraulic main will be considered.

In Fig. 60 the resultant pressure tending to blow the pipe into two portions is $P = q \times D \times L$, where

q = Pressure in pounds per square inch.

D = Diameter of the thin shell or pipe in inches.

L = Length of the portion considered, in inches.

If S is the tensile stress in pounds per square inch of the metal, then the resistance offered by a seamless pipe would be $S \times$ area of metal, i.e., $StL \times 2$, where t = pipe thickness in inches.

Hence $2StL = qDL$ or $S = \frac{qD}{2t}$ 1

If one end of the cylindrical shell is closed the total pressure acting on it is $q \times \text{area of end} = \frac{q\pi D^2}{4}$. This is resisted by the annular ring of metal shown hatched in Fig. 60 (c). The area of this ring is approximately πDt , and its resistance is πDtS .

$$\text{Hence} \quad \pi DtS = \frac{q\pi D^2}{4}, \quad \text{or} \quad S = \frac{qD}{4t} \quad \dots \quad 2$$

The stress of equation 2 is half that of equation 1, so that the circumferential stress is half the longitudinal stress, and therefore circumferential joints need only be half the strength of the longitudinal joints.

The plates for the pipes are edge-planed with a bevel to facilitate caulking. They are then rolled or bent to cylindrical form and riveted longitudinally. The various small cylinders, each from 8 to 15 or 20 ft. long, are so arranged that every second cylinder is slightly larger in diameter than its immediate neighbours. This permits the smaller diameter pipes to be threaded into the larger and then riveted up circumferentially.

Circumferential joints are single-riveted lap joints, while the longitudinal ones are also lap joints, and are either single or double riveted as required. Butt joints are seldom used in the fabrication of mild steel hydraulic mains.

Numerical Example. A hydraulic main 36-in. internal diameter is subjected to a maximum internal pressure of 200 lb. per square inch. Design the longitudinal joint. Take the permissible tensile stress on mild steel as 7 tons per square inch for this class of work.

Since a joint weakens a plate, the strength of the joint is the strength of the pipe. Let the efficiency of a double-riveted lap joint be E per cent. and T the required thickness of plate, then the effective

thickness of the plating T is only $\frac{E}{100}$ of T . If it were possible to

have a seamless pipe of the same diameter, then the thickness of this pipe would only require to be t , i.e., t is less than T .

$$\therefore \quad \frac{E}{100} \times T = t = \frac{qD}{2S}, \quad \text{or} \quad T = \frac{100 qD}{2ES}$$

$S = 7 \times 2,240$ lb. per square inch.

E = an efficiency of approximately 70 per cent. for double-riveted lap joints.

$$\text{Hence} \quad T = \frac{100 \times 200 \times 36}{2 \times 70 \times 7 \times 2,240} = 0.328 \text{ in.}$$

A $\frac{5}{16}$ -in. plate is 0.3125 in., but as $\frac{1}{16}$ in. is often added to the thickness, so as to lengthen the life of the pipe in view of corrosion, the metal adopted would be of $\frac{3}{8}$ -in. plating (0.375 in.).

After a tentative trial the riveting was fixed at $\frac{3}{4}$ -in. diameter at $2\frac{1}{4}$ -in. pitch. The efficiencies work out at:—

$$\text{Plate : } \frac{p - d}{p} = \frac{2\frac{1}{4} - \frac{3}{4}}{2\frac{1}{4}}, \quad \text{or } 66.7 \text{ per cent.}$$

$$\text{Shear : } \frac{1.0472d^2}{pt} = \frac{1.0472 \times \frac{3}{4} \times \frac{3}{4}}{2\frac{1}{4} \times \frac{3}{8}}, \quad \text{or } 69.8 \text{ per cent.}$$

$$\text{Bearing : } \frac{8d}{3p} = \frac{8 \times \frac{3}{4}}{3 \times 2\frac{1}{4}}, \quad \text{or } 88.8 \text{ per cent.}$$

The other dimensions adopted would be: edge of plate to rivet line = $1\frac{1}{4}$ in. (minimum $1\frac{1}{2}d$), rivet line to rivet line = $2\frac{1}{4}$ in. (minimum for diagonal pitch and tearing is $\frac{7}{8}$ of $p = 1.97$), and rivet line to plate edge = $1\frac{1}{4}$ in.

SECTION WEAKENED BY ONE RIVET HOLE

Fig. 61 represents a very common type of joint. Bridge suspenders and tension diagonals were frequently made of flat bars, but this practice is not nearly so common now as formerly. It is an interesting example of how an "engineering trick" can save material, if the assumption as to stress distribution is correct.

Numerical Example. A general drawing shows a flat bar 13" wide $\times \frac{7}{8}$ " thick as being jointed or spliced at a certain point, but no information is given save that the section is to be weakened by only one rivet hole. Since neither the working stresses are given nor the number of rivets, the designer must fall back upon the fact that $F_{ds} = 2F_s = F_b = \frac{4}{3}F_t$. Design the connection.

Diameter of Rivet. Unwin's rule, $d = 1.4\sqrt{t}$ or $1.2\sqrt{t}$, gives a diameter of 1.3 in. or 1.1 in.; the other rule, d not less than t , gives a $\frac{7}{8}$ -in. diameter rivet. Adopt the usual outsize of rivet, namely, 1 in. diameter.

Rivet Values. Considering bearing alone. The sum of the thicknesses of the covers will be greater than the main plate thickness. Since the covers supply the reaction to the main plate both the latter and the former will have the same load to carry, and, therefore, if bearing failure occurs it will do so at the main plate.

The rivets are in double shear and bearing on a $\frac{7}{8}$ -in. thick plate.

Double shear value of a 1" diameter rivet

$$= 0.7854 \times \frac{4}{3}F_t = 1.05 F_t \text{ tons.}$$

Bearing value of a 1" diameter rivet on $\frac{7}{8}$ " Pl.

$$= 1" \times \frac{7}{8}" \times \frac{4}{3} F_t = 1.17 F_t.$$

Method of Failure 1. All the rivets through one main plate will give way by double shear, since the rivet is weaker in double shear than in bearing.

Net area of bar at *A* $= (13" - 1") \times \frac{7}{8}" \text{ tk.} = 10.50 \text{ sq. in.}$

Tensile load of bar at *A* $= 10.5 \text{ sq. in.} \times F_t^T / \text{sq. in.} = 10.5 F_t \text{ tons.}$

No. of rivets required $= 10.5 F_t \div 1.05 F_t = 10.$

Net area of bar at *B* $= (13" - 2") \times \frac{7}{8}" \text{ tk.} = 9.63 \text{ sq. in.}$

" " " " *C* $= (13" - 3") \times \frac{7}{8}" \text{ tk.} = 8.75 \text{ " "}$

" " " " *D* $= (13" - 4") \times \frac{7}{8}" \text{ tk.} = 7.88 \text{ " "}$

The rivets are arranged as in diagram for the following reason.

Method of Failure 2. If the main plate failed by tearing at *B* the left portion could not leave the cover plates until it had double-sheared the rivet at *A* as in Fig. 62. With a tear at *C* there are three rivets to double-shear, viz., one at *A* and two at *B*, before the main bar can travel to the left.

The total strength of the joint at any point on the main plate is thus the sum of the net tensile strength at that section plus the

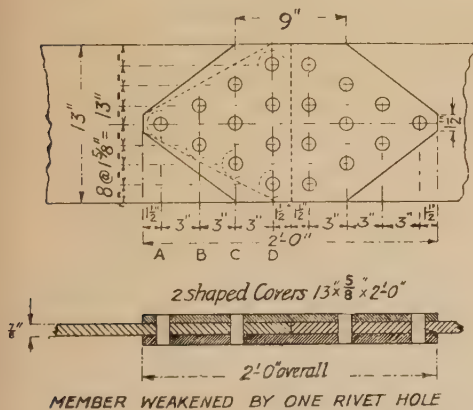


FIG. 61

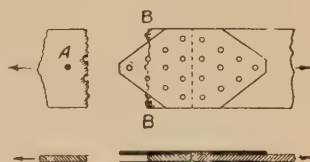


FIG. 62

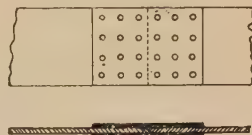


FIG. 63

value of the rivets (either in double shear or bearing, whichever is the lesser) between the fracture and the end of the cover.

Strength at *A* $= 10.50 F_t \text{ tons.}$

" *B* $= 9.63 F_t + 1.05 F_t = 10.68 F_t \text{ tons.}$

" *C* $= 8.75 F_t + 3 \text{ rivets @ } 1.05 F_t = 11.90 F_t \text{ tons.}$

" *D* $= 7.88 F_t + 6 \text{ rivets @ } 1.05 F_t = 14.18 F_t \text{ tons.}$

Method of Failure 3. Covers tearing straight across on the line of the four rivet holes at *D*.

The tensile value of the two covers should be together equal to that of the main plate at *A*.

$$\therefore 2(13'' - 4'') \times t'' \times F_t = 10.5F_t \text{ (where } t = \text{cover thickness),}$$

$$\text{whence } t = 0.583''.$$

It cannot be definitely stated that each cover takes half the pull on the main bar, although we assume this. To make allowance for any eccentricity specifications ask that 5 per cent. more cover area be given than is actually necessary. The thickness *t* of each cover is thus increased to 0.61 in. Adopt $\frac{5}{8}$ -in. cover (0.625 in.).

Joint Efficiency. Strength of plate not holed = $13'' \times \frac{7}{8}'' \times F_t$ tons/sq. in. = $11.375F_t$, whence efficiency = $10.5F_t \div 11.375F_t$, i.e., 92 per cent.

If three rows of four rivets per row had been adopted on each side of joint, Fig. 63, the efficiency would have been $7.88F_t \div 11.375F_t$, or approximately 69 per cent., failure taking place by tearing: thus by giving more rivets the joint may be made considerably weaker.

Shape of Cover. The covers are tapered, but never encroach on the $1\frac{1}{2}d$ distance from the rivet centres, as indicated by the broken line circles round several of the rivets. The plate could have been closer cut after the manner of the dotted outline on the left-hand side, but this makes the joint look too sharp.

The cover is tapered or shaped because at the end it picks up one rivet value of strength, and should have therefore a width of plate

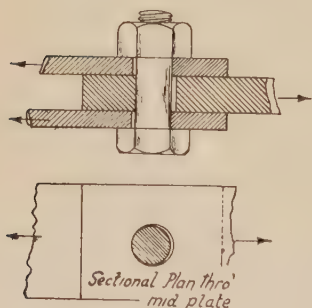


FIG. 64

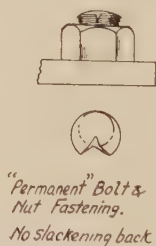


FIG. 65

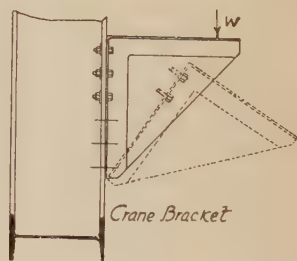


FIG. 66

to correspond in tensile value. The more rivet values the cover picks up from the main plate the wider becomes the cover in proportion.

BOLTS AND NUTS

These are generally used in structural work as temporary fastenings to bind the material together during manufacture.

In the case of the ordinary manufactured mild steel bolt and nut (or black bolt as it is termed) the shank is $\frac{1}{16}$ in. less in diameter than the hole which it enters. As Fig. 64 plainly shows, the shank has only line contact throughout its length against the sides of the hole, instead of surface contact as with the machine-closed rivet. Because of this there will be more "give" in a structure when bolts are employed than when rivets are used. It will be deduced, therefore, that where the shank is subjected to shear and bearing the rivet is the more suitable of the two fastenings. Further, since the bolt does not completely fill the hole, it will have more room to deflect and the bending action of Fig. 53 will be intensified.

Nuts, when screwed up without any special precaution, are apt to work loose if the structure is exposed to any vibration. Where vibration is present the part of the bolt projecting beyond the nut is hammered out, and in that way the nut and bolt are locked together. This splaying out of the $\frac{1}{4}$ -in. projection is achieved either by cold knobbling—*i.e.*, burring up with a hammer—or by cutting a longitudinal V-shaped chase or groove across the screw threads with a cold chisel. If the upper bolts of the crane bracket connection, Fig. 66, are so treated there will be no slackening back of the nuts. Fig. 65 shows the effect of the V groove.

In the connections of roof girders to columns and roof truss shoes to columns or to roof girders, bolts are often used, *i.e.*, in the connection of one portion of a structure to another, but not in the individual elements composing each portion.

Roof truss members are shop-riveted where possible and the truss sent to the site in as few parts as practicable. These parts are field-riveted or bolted at ground level on the site and the completed truss lifted up to its final position, either with an erection pole or crane, and the end connections bolted. These latter connections are high up in the air, and in all probability bolts would give as good a job as rivets because of the difficulty of satisfactory riveting. If there are only a few holes in each connection no scaffolding would be erected. The rivet forge would be at ground level and the rivets sent up in a bucket, pulled hand over hand by a rope, to the detriment of the rivet, which would be cooled. In cases of this kind bolts are permitted, in fact preferable. Separate working stresses for bolts may be used, but another method is to calculate the number of shop rivets required and this increased by 30 to 50 per cent. (according to different specifications) gives the number of black

bolts. See Vol. II for lists of working stresses on mild steel for different types of structures ; also refer to page xv of this volume.

Bright, fitted or turned bolts are machine-turned mild steel bolts and are made to give a tight fit at entry, with a maximum clearance of 0.005 in. In contradistinction to black bolts, they are always provided with washers. They are therefore much superior to black bolts, and are taken as being equivalent to shop rivets of the same diameter.

Fastenings under Tension. Bolts should preferably be used where there is a possibility of longitudinal or axial tension, as in the upper bolts of Fig. 66. In this example the fastenings at the top have a force exerted upon them tending to pull off their heads. The resisting area is the area at the bottom of the thread of the bolt. When wrought iron rivets were used it was by no means exceptional to have a pressure machine-closed rivet head fly off on cooling after being riveted up ; a failure which occurs but seldom with steel rivets. To appreciate this point thoroughly consider the range of temperature which a rivet may have. From ordinary temperature up to a light cherry-red heat represents, say, a range of 1,500° F. A rod 2.973 in. long on heating through this range expands to 3 in., and conversely a 3-in. rod at light cherry-red contracts on cooling to 2.973 in.

The pull necessary to extend a rod 2.973 in. long up to 3 in. is found from the following :—

Young's modulus, $E = \text{Stress} \div \text{Strain}$,
whence $\text{Stress} = E \times \text{Strain}$.

If $E = 13,000$ tons per square inch,

$$\begin{aligned} \text{then} \quad \text{Stress} &= 13,000 \times \frac{3 - 2.973}{2.973} \\ &= 118 \text{ tons per square inch.} \end{aligned}$$

This is not a fair comparison perhaps, but nevertheless it illustrates in a striking manner that the rivet shank must be under high axial tension. The pulling inwards of bulging walls of unstable buildings is accomplished in the same manner, *viz.*, by heating the tie rod and screwing up tight when the rod is expanded. On the rod cooling it contracts and exerts an inward pull on both walls.

Leading authorities are at variance on the question of permitting rivets to carry axial tension. Some forbid their use entirely, while others permit it provided a low tensile working stress is used. Thus the list of Rivet and Bolt Values for Buildings, page xv, gives the following : $\frac{4}{3}F_t$ for site rivets, $\frac{5}{3}F_t$ for shop rivets, $\frac{5}{3}F_t$ for bright and black bolts less than $\frac{3}{4}$ in. dia., and $\frac{6}{3}F_t$ on all bolts $\frac{3}{4}$ in. dia. and up. (The net area of a bolt is the area at the bottom of the thread.)

Even in a bolted connection the bolt is not free from initial tensile stress. The tight screwing up of the rod tends to elongate the shank and stress must occur in proportion to the strain produced. There is, in addition to this, a slight torsional effect. This screwing up stress is more dangerous in small diameter than in large diameter bolts; in fact a small bolt can be snapped in two by overtight screwing. By limiting the lever arm of the spanner to $15d$ a rough control is exercised upon the workman.

Rule for Area at the Bottom of the Thread.* Convert the bolt diameter into eighths of an inch, then subtract one-eighth from this total. These two numbers when multiplied together and divided by 100 give a very close approximation to the required area.

Thus 1-in. diameter bolt, area at bottom of thread $= 8 \times 7 \div 100 = 0.56$ sq. in. (correct area $= 0.554$); $\frac{7}{8}$ -in. diameter bolt, area at bottom of thread $= 7 \times 6 \div 100 = 0.42$ sq. in. (correct area $= 0.422$).

Foundation Bolts are usually larger in diameter than the ordinary fastenings, and are about 24 diameters long, see Chap. III. Vol. II. They may have to take uplift, *i.e.*, they are in tension, and in addition may have to carry shearing and bearing forces, or a combination of these. Washers are always used with foundation bolts, as the holes have a clearance of $\frac{1}{8}$ in. or $\frac{1}{4}$ in. over the bolt diameter. The washer helps in spanning the hole and in distributing the load on to the nut. These bolts are in the majority of cases grouted in with cement, as also the nuts, or they are covered with concrete, so that there is little possibility of their becoming undone, intentionally or otherwise, the concrete serving as a lock nut.

Lock Nuts may be used instead of knobbing and chasing, but are seldom used in constructional work.

Tapered Washers are necessary where the outer member,

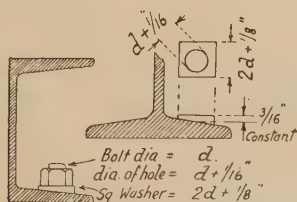


FIG. 67

penetrated by a bolt, has a splayed surface. Fig. 67 illustrates their use.

* Rule by Mr. W. Stevenson on p. 384 (eighth edition), *Goodman's Mechanics Applied to Engineering*.

Shop and Field Rivets. As is to be expected from what has been said, the rivet completed in the shop is a much superior rivet to that driven at the site. Specifications differentiate sharply between these two types of rivets by requiring that rivets if site-driven are to have their number increased by 20 to 25 per cent. over the number which would have been necessary had they been shop rivets.

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CHAPTER V

FLANGE PLATE SPLICES

Zigzag Tearing and Net Areas. A student is apt to lay great stress upon the fact that a flat bar with a central hole drilled in it fails, under axial tension, at a higher load per net square inch than a similar bar without a hole. This is due to the help given by the supposed non-active metal behind the hole, parallel to the direction of pull. Just prior to breaking, the metal tends to narrow at that point where the fracture occurs, and the surplus metal behind the hole restricts this contraction of the strained metal and thus increases the ultimate strength. When there are several holes out of a member they are generally reeled, with perhaps only two rivets out at any cross-section of the bar, as in Fig. 68 (a). Now the stress

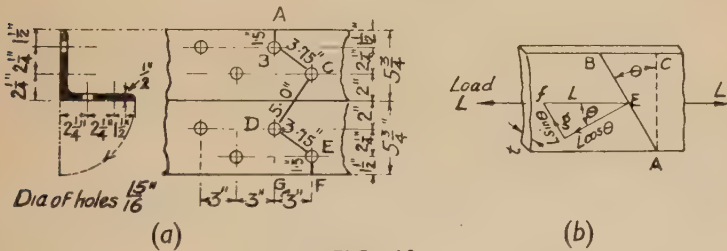


FIG. 68

in the bar can hardly be expected to jump from side to side in order to clear the holes without some loss of strength, so that the assumed gain mentioned at the commencement of the paragraph can scarcely be counted upon. Further, it brings in that debatable point as to whether the elastic limit or the ultimate strength should be used when fixing the working stresses.

Fig. 68 (b) is one which finds a place in nearly every text-book upon strength of materials. The load L is axial on the flat bar, whose width is AC and thickness is t . The tensile load per square inch on

$$AC = L \div AC \times t \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad 1$$

Resolving the load L normal to and along the diagonal plane AB , whose length $= AC \sec \theta$, then the shear along AB , per square inch,

$$= L \sin \theta \div AC \sec \theta \times t = L \sin \theta \cos \theta \div AC \times t \quad . \quad 2$$

Tensile stress normal to AB , per square inch

$$= L \cos \theta \div AC \sec \theta \times t = L \cos^2 \theta \div AC \times t \quad . \quad . \quad \mathbf{3}$$

Value **3** can never be greater than that of **1**, since $\cos \theta$ never exceeds unity, but plane AB , although it has a smaller normal tensile stress on it than AC , has the additional shearing stress.

Experimental results are rather erratic, but they all tend to show that a bar with reeled holes has a distinct penchant for tearing along the zigzag lines instead of straight across by the shortest route. That is, it tends to tear with the grain of the metal. When the diagonal line has a net tearing length (*i.e.*, between the holes and not centre to centre of holes) of about 30 per cent. in excess of the net straight across length, the tear occurs as readily one way as the other. If $AC = 100$, say, and $AB = 130$, then the amount to be allowed for AB by the B.S.S. is 0.8 of 130, *viz.*, 104, thus meeting the experimental evidence.

The figures given for the pitches of Table 11 and for the areas of Table 12, Appendix, to meet the requirements of the B.S.S. on zigzag tearing, were obtained by assuming the angles flattened out, as in Fig. 68(a), so as to give equivalent flat bar sections.

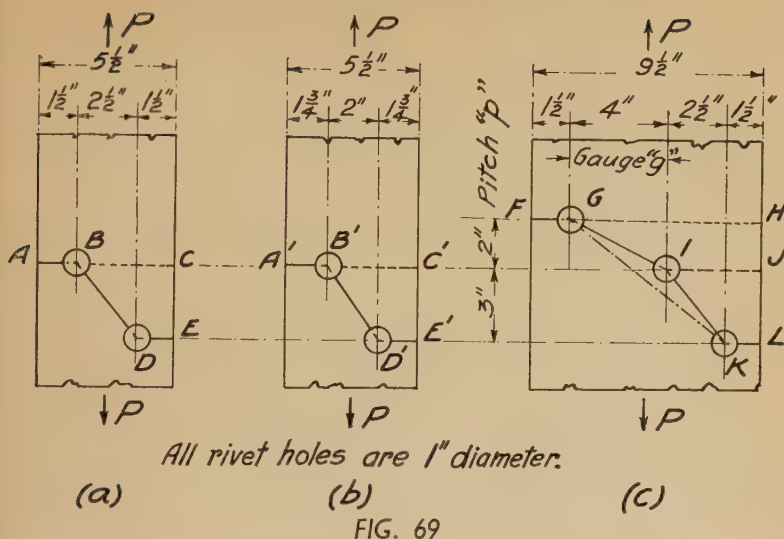
Arithmetical Examples : British Specification. Fig. 69

The lengths of the diagonal paths, centre to centre of rivet holes, are :— $BD = 3.9''$; $B'D' = 3.6''$; $GI = 4.47''$; $IK = 3.9''$ and $GK = 8.2''$.

- (a) Path ABC :— Net width = $5.5'' - 1'' = 4.5''$
 Path $ABDE$:— $(1\frac{1}{2}'' - \frac{1}{2}'') + 0.8 (3.9'' - 1'') + (1\frac{1}{2}'' - \frac{1}{2}'') = 4.32''$
 (b) Path $A'B'D'E'$:— $(1\frac{3}{4}'' - \frac{1}{2}'') + 0.8 (3.6'' - 1'') + (1\frac{3}{4}'' - \frac{1}{2}'') = 4.58''$
 (c) Path FGH :— Net width = $9.5'' - 1'' = 8.5''$
 Path $FGIJ$:— $(1\frac{1}{2}'' - \frac{1}{2}'') + 0.8 (4.47'' - 1'') + (4'' - \frac{1}{2}'') = 7.28''$
 Path $FGIKL$:— $(1\frac{1}{2}'' - \frac{1}{2}'') + 0.8 (4.47'' - 1'') + 0.8 (3.9'' - 1'') + (1\frac{1}{2}'' - \frac{1}{2}'') = 7.1''$
 Path $FGKLL$:— $(1\frac{1}{2}'' - \frac{1}{2}'') + 0.8 (8.2'' - 1'') + (1\frac{1}{2}'' - 1'') = 7.76''$

The net area of a plate is the least path length multiplied by the plate thickness.

American Specifications make use of an empirical formula which embodies the results of an intensive research into diagonal tearing. If the rivet pitch p be measured along the length of the member and the "gauge" g , or transverse spacing, be measured across the member, then the value of $p^2 \div 4g$ is estimated for each pair of holes. When considering diagonal tearing subtract the sum of the diameters of all the holes on the path from the gross width of the member and to this result add the quantities $p^2 \div 4g$ for each pair of rivet holes on the path to give a value which is used as the



equivalent net width. The following three arithmetical examples will make the method clear.

(a) Path ABC :—Net width $= 5.5'' - 1'' = 4.5''$
 Path $ABDE$:—Subtract 2 holes $= -2''$
 Add $p^2/4g = 3''^2/4(2\frac{1}{2}'')$ $= +0.9''$
 Equivalent net width $= 5.5'' - \underline{1.1''} = 4.4''$

(b) Path $A'B'C'$:—Net width $= 5.5'' - 1'' = 4.5''$
 Path $A'B'D'E'$:—Subtract 2 holes $= -2''$
 Add $p^2/4g = 3''^2/4(2'')$ $= +1.125''$
 Equivalent net width $= 5.5'' - \underline{0.875''} = 4.625''$

It will be noted that the total net deduction for diagonal tearing decreases as rivet D approaches the vertical through B . In the limit, when D is vertically below B , there is no diagonal tearing but only two identical transverse paths, each of $4.5''$ net width.

(c) Path FGH :—Net width $= 9.5'' - 1'' = 8.5''$
 Path $FGIJ$:—Subtract 2 holes $= -2''$
 (GI), Add $p^2/4g = 2''^2/4(4'')$ $= +0.25''$
 Equivalent net width $= 9.5'' - \underline{1.75''} = 7.75''$

Path <i>FGIKL</i> :—Subtract 3 holes	=	—3"
(GI), Add $p^2/4g=2''^2/4(4'')$	=	+0.25"
(IK), Add $p^2/4g=3''^2/4(2\frac{1}{2}'')$	=	+0.9"
Equivalent net width	=9.5" —	<u>1.85"</u> =7.65"

Path <i>FGKLL</i> :—Subtract 2 holes	=	—2"
(GK), Add $p^2/4g=5''^2/4(6\frac{1}{2}'')$	=	+0.96"
Equivalent net width	=9.5" —	<u>—1.04"</u> =8.46"

In example (c) the net tensile area of the member is $7.65'' \times$ thickness t of the plate. In all the examples the American method is the more generous.

Length of Flange Plates. The edges of flange plates (if so specified) are planed by machines, which are generally anything from 10 ft. to 30 ft., or even 40 ft. long. The machines usually, or should, have open ends, so that if a plate is longer than the machine it can be planed throughout its full length by pulling it along and resetting it. The plate is held in position by vertical jacks—screw or hydraulic—varying in number up to about a dozen, depending upon the length and make of the machine. These jacks have to be unloosened, the plate shifted by crane and trued up anew, and, finally, the jack pressure re-exerted. The time spent in the foregoing operations is much greater than the actual time spent in the machining.

Further, it was noted in Chapter I., under the heading of extras charged on plates, that long narrow plates could not be sheared with sides exactly parallel. The longer the plate the greater is the twist and, consequently, the amount of planing required is larger.

General British practice has thus limited the length of flange plates to about 30 ft., and splicing them if they exceed this length, the extra material involved in the covers being more than paid for by the time saved in handling the shorter plates.

In America the practice is to specify that the inner flange plate, adjoining the flange angles, "shall run the full length of the girder continuously, without being spliced." Thus the $12'' \times \frac{7}{16}''$ plate of Fig. 70, if it occurred in a girder of 50 ft. length, would be unbroken, whereas in British practice the plate might be spliced at 17 ft. from each end, or, alternatively, say at the centre of the span, or some other suitable position, depending entirely upon the design and the designer.

No fixed or definite rule can be laid down regarding lengths of flange plates, and the designer settles each job upon its own merits. Theoretically, try to do without splices, but, from a practical point

of view, remember cost. Since practice limits the length as stated, it may be taken as axiomatic that a loss results from the use of long, narrow plates.

Single Flange Plate Splice (Tension Flange). Given a $12'' \times \frac{7}{16}''$ flange plate, Fig. 70, and using $\frac{3}{4}$ in. diameter rivets, design the joint.

Specifications require that the plate shall be covered fully to develop the effective strength of the member. The effective strength = effective area (net in this case) $\times$ permissible stress. A transverse sectional area, as from a to e , will be assumed as being the effective area, and when the pitch has been fixed the question of the possibility of diagonal tearing will be examined. It is assumed that no help may be obtained from the main angles and web plate as these, presumably, are working at their full capacity.

Looking at the small single line diagram, it will be seen that failure may occur in two ways: firstly, by tearing of the cover plate at the joint, thus allowing the main plate to fly apart; secondly, through failure of the rivets either in bearing or in single shear as illustrated.

Rivet Values.

One $\frac{3}{4}$ -in. diameter rivet in *S.S.* = $\frac{2}{3}F_t \times 0.44 = 0.29F_t$ tons.

One $\frac{3}{4}$ -in. diameter rivet in bearing on $\frac{7}{16}$ -in. plate = $\frac{4}{3}F_t \times \frac{3}{4}'' \times \frac{7}{16}'' = 0.44F_t$ tons.

The lesser of these two values is single shear and the rivets will fail in *S.S.* in preference to bearing.

Plate Value, etc.

The area of the cover plate must be 10 per cent. in excess of the main plate because it is an unsymmetrical cover.

Transverse path, a, b, c, e = $12'' - 2 @ \frac{3}{4}'' = 10.5$ in.

Transverse net area = $10.5'' \times \frac{7}{16}'' = 4.59$ sq. in.

Transverse strength = $4.59 \times F_t = 4.59F_t$ ton.

No. of rivets required = $4.59F_t \div 0.29F_t = 16$

(i.e., 16 on each side)

Cover net area required = $4.59 + 10\% = 5.05$ sq. in.

Adopt $\frac{1}{2}''$ cover, net area = $10.5'' \times \frac{1}{2}'' = 5.25''$

Cover plate and riveting are each as strong as the main plate.

The rivets are closely pitched in order to keep the cover plate short; the pitch adopted is $2\frac{1}{2}$ in., which is larger than the minimum of $3d = 2\frac{1}{4}$ in.

Diagonal Tearing. The $12'' \times \frac{1}{2}''$ cover plate may tear diagonally along the zigzag line a, b, f, g whose effective length in accordance with the standard specification is

$$(ab - \frac{1}{2}d) + 0.8 (bf - d) + (fg - \frac{1}{2}d) = 3.44'' + 3.4'' + 3.44'' = 10.28''$$

Where d is the diameter of the rivet, viz. $\frac{3}{4}$ in.

Net transverse length $a e$ is $12'' - 2 @ \frac{3}{4}'' = 10.5''$.

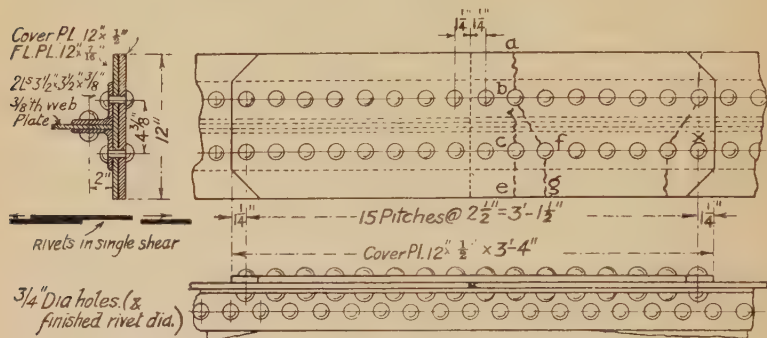


FIG. 70

The effective area of the cover plate against tearing = $10.28'' \times \frac{1}{2}'' = 5.14$ sq. in., i.e., more than the required 5.05 sq. in. The pitch, therefore, does not require to be increased because of diagonal tearing. The zigzag tearing, mentioned above, will occur right over the joint of the flange plate, but for clearness the line $a b f g$ has been shown to the right of the joint.

Should the $12'' \times \frac{7}{16}''$ main plate tear diagonally it will do so near the end of the cover plate at X , where it is not fully reinforced by the cover plate. This is assuming, of course, that the rivets are closely pitched only in the length of the cover, and that when beyond the cover the rivets are spaced far enough apart to ensure transverse tearing taking place. The effective area of the main plate is the diagonal effective length $\times$ thickness

$$= 10.28'' \times \frac{7}{16}'' \text{ net} = 4.5 \text{ sq. in.}$$

But before the main plate can fly apart it must shear the rivet at X .

(This will be single shear since, according to our original assumption, the angles are virtually non-existent.)

The total value of the main plate and rivet at X is thus $4.5F_t + 0.29F_t = 4.79F_t$, i.e., greater than the main plate's standard value of $4.59F_t$ tons.

Since the rivets in the main angles of the flange reel with those in the flange plate, the net area of the tension flange angles would require to be checked for diagonal tearing due to close pitching of

the rivets in the length of the splice. However, if possible, a flange splice is never placed near the point of maximum bending moment, but always some distance away from this point. There is, therefore, the probability of an excess of flange area at the splice, which permits the designer to take away a small amount of the flange angle area owing to the close pitching of the rivets.

The rivet pitch can be opened out after passing the flange plate splice, so that at the point of maximum bending moment (and, therefore, minimum shear) the pitch is wide, and the net transverse area, and not the zigzag area, will be the minimum.

Single Flange Plate Splice (Compression Flange). Main sections as for tension flange.

The practice of several firms used to demand that the splice of a compression member should be designed as if that member were in tension and not in compression. From this point of view the above joint would do for both tension and compression flanges, and its use in the compression flange would certainly fulfil the requirements of those specifications which demand that the effective strength of the member be developed.

Plate Value, etc.

Gross area of plate $12'' \times \frac{7}{16}'' = 5.25$ sq. in.

Effective strength of main plate = gross area

$\times$ permissible compressive stress $F_c = 5.25F_c$ tons.

S.S. value of $\frac{3}{4}''$ dia. rivet $= 0.29F_t$ tons.

Number of rivets required to develop plate

$$= 5.25F_c \div 0.29F_t = 18F_c \div F_t.$$

Cover, gross area required $= 5.25 + 10\% = 5.78$ sq. in.

Cover plate adopted $12'' \times \frac{1}{2}''$, gross area of which $= 6.00$ sq. in.

The permissible F_c will be known from the design of the main girder flanges, and when substituted in the above equations will give the required number of rivets.

Checking the Tensile Joint used as a Compressive Joint. In Chapter I. of Vol. II. it is given that the working stress on the gross cross-sectional area of a plate girder flange with unstiffened edges is $F_c = F_t (1 - 0.01 l/b)$; where l is the unsupported length and b is the breadth of the flange. A fairly common value for l/b is 20, so that the corresponding value for F_c is $F_t (1 - 0.01 \times 20) = 0.8F_t$. Hence the number of rivets required is $5.25F_c \div 0.29F_t = 5.25 \times 0.8F_t \div 0.29F_t = 14\frac{1}{2}$ rivets.

The net or tension method gave 16 rivets, and is therefore on the safe side.

The cover plate of $\frac{1}{2}$ in. found by the tensile method is also

sufficient from the point of view of compression. Diagonal tearing does not require to be considered in a compression member, and the design is simplified accordingly. In the compression member the rivet pitch should, therefore, be as near the minimum as possible in view of the strut action of the individual plates, and also to shorten the cover plate for economy.

Alternative Single Flange Plate Splice (Tension). Fig. 71 is an alternative design to that of the previous figure.

The distance between the toe of the flange angle and the edge of the flange plate is $2\frac{5}{16}$ in. The rivets are $\frac{3}{4}$ in. diameter, so that if cover strips are used on the inside face of the flange plate the minimum width requires to be $3d$, i.e., $2\frac{1}{4}$ in. By using a flat or plate of $2\frac{1}{4}$ in. width a clearance of $\frac{1}{16}$ in. is obtained at the toe of the main angles, and any irregularities of the angles will be cleared. Appearance fixed the thickness of these strips at $\frac{3}{8}$ in. to agree with the flange angle, and a flush surface is thus presented to the rivet die. This type of detail could not have been used if the flange plate had been narrower than 12 in.

The rivet pitch adopted is 6 in. (i.e., 3 in. alternate pitch), because there are now four rivet lines along the plate offering a greater possibility for diagonal tearing than in the previous case of two rivet lines. With this width of pitch there is only transverse tearing of the flange angles. It is assumed that the girder shear calculations permit of a 6-in. pitch in the angles.

Covers tearing at the weakest section, i.e., over the joint. Main cover is $\frac{7}{16}$ in. thick, i.e., same thickness as the main plate.

Net transverse area of cover and two strips

$$= (12'' - 2 @ \frac{3}{4}) \frac{7}{16}'' \text{ plus } 2 @ 2\frac{1}{4}'' \times \frac{3}{8}'' = 6.28 \text{ sq. in.}$$

Diagonal tearing of outer cover *a b c e f g*

$$= 10.32'' \times \frac{7}{16}'' = 4.52 \text{ sq. in. net.}$$

Plus transverse tearing of strips

$$\text{at } a \text{ and } g = 2 (2\frac{1}{4}'' - \frac{3}{4}'') \frac{3}{8}'' = 1.12 \text{ sq. in. net.}$$

$$\text{Total} = 5.64 \text{ sq. in.}$$

This more than satisfies the required cover area of 5.05 sq. in. of the previous calculations.

Assuming the angles to be non-existent, the rivets connecting the inner cover strips, flange plate, and outer main cover are in double shear ($0.58F_t$ per rivet) or bearing on the mid $\frac{7}{16}$ -in. flange plate ($0.44F_t$ per rivet).

Rivet Strength. Ten rivets in *S.S.* + 4 in bearing = $10 \times 0.29F_t$ + $4 \times 0.44F_t = 4.66F_t$ tons, which is satisfactory, the effective main plate strength being $4.59F_t$ tons.

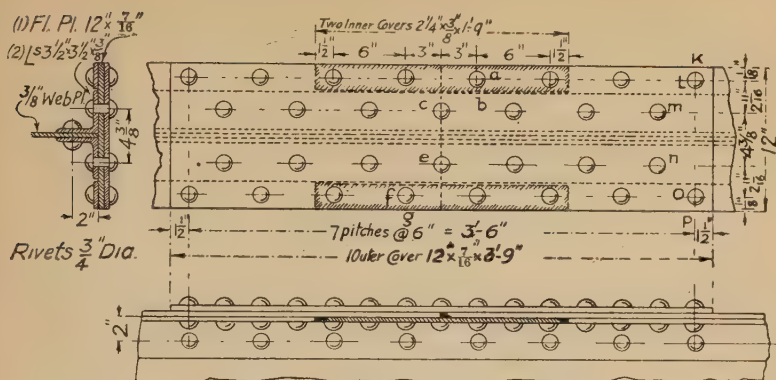


FIG. 71

No help is given by the two rivets on the line of the joint.

The rivets connecting the narrow strips to the flange plate are in double shear, made up of one single shear between the inner strip and flange plate, plus another single shear between flange plate and the outer cover. The inner strips are, therefore, attached by rivets which, so far as the individual strips themselves are concerned, are in single shear. The rivets connecting the strips to the flange plate should just develop the strength of each strip on each side of the joint.

$$\text{Net area of one strip} = (2\frac{1}{4}'' - \frac{3}{4}'') \times \frac{3}{8}'' = 0.5625 \text{ sq. in.}$$

$$\therefore \text{Number of rivets required to develop the}$$

$$\text{strip} = 0.56F_t \div 0.29F_t \text{ (i.e., } S.S. \text{ value)} = 2$$

Occasionally, however, the strips are carried the full length of the outer cover for appearance, but no extra strength is gained thereby as the plates would fail by tearing, while the connecting rivets would remain intact. Theoretically, both rivets and plates should have the same ultimate strength.

Main Flange Plate Tearing Diagonally. This would occur at the weakest part, *k l m n o p*. The right portion of the flange plate would then travel towards the right hand by slipping out from under the outer cover and leaving it undisturbed. If the tear occurs nearer the joint there will always be some rivets to shear before the flange plate frees itself from the outer cover.

Effective diagonal area of flange plate

$$= \text{diagonal length} \times \text{thickness}$$

$$= 10.32'' \times \frac{7}{16}'' = 4.52 \text{ sq. in. net,}$$

which is slightly less than the net transverse area of 4.59 sq. in.

Spacing the end four rivets wider apart longitudinally than the others in the cover plate would tend to prevent diagonal or zigzag tearing of the flange plate, provided, of course, that the rivet pitch beyond the joint is wider than that within the spliced portion. On the other hand, the rivets at the ends of the covers are more highly stressed than the ones nearer the joint, and for this reason the rivets, theoretically, should be clustered near the end of the cover. By Hooke's law, strain or elongation is proportional to the stress or force, and therefore at $l o$ in the main plate there will be maximum stretch since the load is at its maximum value. After $l o$ has been passed the stress in the main plate has been lowered by two rivet values, which have been taken from the main plate and transferred into the cover. Similarly, after $m n$ the stress in the main plate is further reduced by other two rivet values. The amount of stretch between each pair of rivets in a longitudinal direction of the main plate decreases towards the centre of the splice. That piece of the main plate which stretches the most will therefore throw a greater load on to its connecting rivets, and this occurs at $l o$.

Practice disregards this anomaly by making all the pitches the same throughout the length of the cover. In this particular case the previous type of splice with no strips is to be preferred as it is simpler and requires less metal.

Special Case of Single Flange Plate Splice. In Fig. 72 the plate which is being covered is one of the inner plates of the flange and not the outer plate. Notwithstanding the fact that other plates intervene between the broken plate and its cover, the design is carried through as if they were in direct contact. The method of procedure is, therefore, exactly similar to that employed in the calculations of the joint illustrated in Fig. 70. Cover plate area = flange plate area plus 10 per cent., while there should be sufficient rivets in the cover plate, on each side of the joint, to develop in single shear the strength of the broken flange plate.

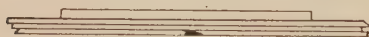


FIG. 72



FIG. 73

The intervening plates are not packers within the usual meaning of the term. The various flange plates are riveted together throughout their full length and may be assumed to form a solid mass of steel, Fig. 73. Into this "monolithic" mass an incision is made at the joint, and to cover this incision, gap, or joint the cover plate is placed in direct contact with the outer flange plate.

It is worthy of note that the American Railway Engineering

Association terms this an indirect splice, since the splice plate is not in direct contact with the parts which it connects. This specification demands that "rivets should be used on each side of the joint in excess of the number theoretically required to the extent of one-third of the number for each intervening plate." This is undoubtedly excessive in this case, as can be proved by following the shears from the angle surface outwards to the cover plate. However, if the grip of the rivets becomes greater than $4d$ the cover plate should be lengthened to include the necessary additional rivets, as previously specified, *viz.*, 1 per cent. increase for each additional $\frac{1}{16}$ in. grip over four rivet diameters in length.

Grouped Flange Plate Splice. In Fig. 74 there are three flange plates all broken near one another and all are covered by one continuous cover plate. Reading from the flange angles outwards, the plates are $20'' \times \frac{9}{16}''$, $20'' \times \frac{9}{16}''$, and the outermost plate $20'' \times \frac{7}{16}''$.

The rivet diameter was fixed at $\frac{1}{4}''$ from the main calculations, and the values per rivet are $S.S. = 0.46F_t$, $D.S. = 0.92F_t$ and bearing on a $\frac{3}{8}$ -in. plate $= 0.47F_t$.

Since all the plates employed are thicker than $\frac{3}{8}$ in. rivet failure will occur in single shear, the least of the rivet values. This mode of failure is illustrated in Fig. 75.

A 3-in. reeled pitch, *i.e.*, a 6-in. pitch, will be used for the flange plate splice for the following reason. From Table 12 (Appendix), sectional areas of $6'' \times 6''$ angles with double rivet lines, it is found that with $\frac{1}{2}$ -in. diameter rivets the zigzag tearing area is not less than the transverse tearing area, and so the splice rivet holes do not weaken the main angles in any way. This assumes, of course, that a closer reeled pitch than 3 in. is not required, which assumption is probably correct, as a simply supported girder will never require a splice of this type near the abutments where the shear is high and, therefore, the rivet pitch close.

Net transverse length *abdeg*h

$$= 20'' - 4 @ \frac{1}{4}'' = 16.25''$$

Zigzag path *abcdefgh*

$$= 2(1.5'' - \frac{1}{2}'' + 2[0.8(4.8'' - \frac{1}{2}'')] + 2[0.8(3.7'' - \frac{1}{2}'')] + (5'' - \frac{1}{2}'') = 16.72''$$

Zigzag path *abcfgh*

$$= 2(1.5'' - \frac{1}{2}'') + 2[0.8(4.8'' - \frac{1}{2}'')] + (9.5'' - \frac{1}{2}'') = 16.8''$$

Should the cover plate fail by tearing at points *W*, *X* or *Y*, it will do so through a transverse section on the line of the four adjacent collinear rivets. Only the transverse tearing of angles and plates need now be expected in the event of failure by tearing.

Each flange plate, as hitherto, will require to be developed by the rivets, which are in single shear in this example.

$20'' \times \frac{9}{16}''$.—Net transverse area = $16.25'' \times \frac{9}{16}'' = 9.14$ sq. in.

$20'' \times \frac{7}{16}''$.—Net transverse area = $16.25'' \times \frac{7}{16}'' = 7.11$ sq. in.

Rivets required for $20'' \times \frac{9}{16}''$ pl. = $9.14F_t \div 0.46F_t = 20$ rivets

Rivets required for $20'' \times \frac{7}{16}''$ pl. = $7.11F_t \div 0.46F_t = 16$ rivets

The unbroken or original strength of the flange plates

$$= 9.14F_t + 9.14F_t + 7.11F_t = 25.39F_t.$$

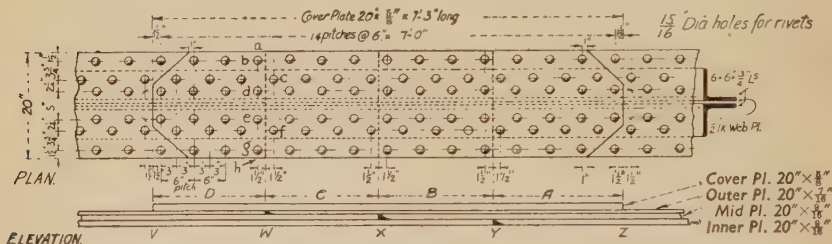


FIG. 74



FIG. 75

Cover Plate. One method of joint failure is for the main plates and cover plates to tear straight through, and this would naturally occur at a point where one of the flange plates has its continuity broken by the joint. The strength in tension of the joint should everywhere be at least equal to the tensile strength of the original two $\frac{9}{16}$ -in. flange plates plus one $\frac{7}{16}$ -in. flange plate.

Thus, tearing at *W*,

$$\begin{aligned} 2 \text{ unbroken } 20'' \times \frac{9}{16}'' \text{ pls.} + \text{cover pl.} &= 2 \text{ pls. } 20'' \times \frac{9}{16}'' + \\ 1 \text{ pl. } 20'' \times \frac{7}{16}'' &\dots\dots\dots 1 \end{aligned}$$

Hence the cover pl. at *W* = at least the $20'' \times \frac{7}{16}''$ pl. + 10 per cent., as the cover is unsymmetrical.

Tearing at *X*

$$\begin{aligned} 1 \text{ unbroken } 20'' \times \frac{9}{16}'' \text{ pl.} + 1 \text{ unbroken } 20'' \times \frac{7}{16}'' + \text{cover} \\ \text{pl.} &= 2 \text{ pls. } 20'' \times \frac{9}{16}'' + 1 \text{ pl. } 20'' \times \frac{7}{16}'' \dots\dots\dots 2 \end{aligned}$$

The cover pl. at *X* must at least = $20'' \times \frac{9}{16}''$ pl. + 10 per cent., because the cover is unsymmetrical.

By similar reasoning the cover pl. at *Y* should develop the $20'' \times \frac{9}{16}''$ pl. + 10 per cent. 3

The plate acting as a cover would not be tapered in thickness to suit 1 and 2, but would be carried throughout at the thicker size necessary.

Cover area required = net area of $\frac{9}{16}$ " pl. + 10 per cent. = 9.14 sq. in. + 10 per cent. = 10.05 sq. in.

Cover area adopted = $20" \times \frac{5}{8}"$, net area of which = $(20" - 4 @ \frac{15}{16}") \times \frac{5}{8}"$ = 10.16 sq. in.

the transverse tearing area being less than the zigzag tearing area.

Rivet Failure. It is assumed that the joint fails, as in Fig. 75, due to the rivets giving way in single shear and that no help is afforded by the main angles which may be regarded as being non-existent.

If joint *W* of the $\frac{7}{16}$ -in. plate were the only joint then the *D* group of rivets = *C* group of rivets in *S.S.* i.e., equal to the strength of the $\frac{7}{16}$ -in. plate = 16 rivets in *S.S.* . 4
as previously explained for the single flange plate splice.

Similarly if *X* were the only joint, then group *C* in *S.S.* = group *B* in *S.S.* = strength of $20" \times \frac{9}{16}"$ pl. = 20 rivets in *S.S.* . 5

And the joint at *Y* would have group *B* in *S.S.* = group *A* in *S.S.* = strength of $20" \times \frac{9}{16}"$ pl. = 20 rivets in *S.S.* . 6

Summarizing the above results and taking the greater where two values occur, groups *D*, *C*, *B* and *A* should have 16, 20, 20 and 20 rivets respectively.

The joint could fail owing to all the rivets giving way simultaneously in single shear as indicated in Fig. 75, i.e., rivet groups *D*, *C* and *B*.

Value = $16 + 20 + 20 @ 0.46F_t$ per rivet in *S.S.* = $25.76F_t$ tons.
The original strength of the flange plates was = $(2 @ 9.14 + 7.11)F_t$ = $25.39F_t$ tons.

The joint is therefore as strong as the original plates, both as regards tearing and rivet failure.

Practice very often makes the rivets in groups *A* and *D* of sufficient number to develop in single shear the net tensile strength of the cover, i.e., 10 per cent. in excess of the strength of the thickest plate covered 7

A chain which can carry a load of 110 tons when fastened by an end hook which can only carry 100 tons is only useful for a 100-ton load. The same argument applies to the cover and its end rivets. The B.S.S. does not specify the extra rivets, but only the extra area.

Each group of rivets in the cover plate should develop the broken plate underneath it by the single shear (or bearing) value of the

rivets. Groups *A* and *B* were given 22 rivets each, while groups *C* and *D* were given 20. It was aimed to give each group 20 rivets, *i.e.*, fully developing the $\frac{9}{16}$ -in. plates, and making group *A* equal to group *D* as a compromise to 7. The riveting did not permit of this, so the extra rivets were given at the heavy end of the joint. The cover plate is cut away at the ends as shown in the plan of Fig. 74, instead of straight across as with Fig. 71. This was done not only for appearance, but also for the following reasons :—

(1) The cover plate area should be increased from the end towards the centre in proportion as the cover plate picks up load from the main plate through the rivets. At the end there are two rivets, therefore a narrow plate ; at the second row of rivets there is a total of $2 + 2$ rivets acting ; while at the third row there are now 8 rivets acting, and therefore a wider or stronger plate. This gives a more evenly applied load to the cover plate than the method of Fig. 71.

(2) It permits of the rivet die getting down to the flange plate rivet heads without fouling the edge or end of the cover plate.

(3) The end edge of the cover plate being closely adjacent to a rivet is held firmly to the main plate and no interstice is left for corrosion.

The dotted lines indicate how the cover ends were “ struck ” out on the drawing.

The above joint presents no difficulty, yet innumerable mistakes are made in its use. Cases are known where the cover plates on the drawings extended from *V* to *X* only, instead of from *V* to *Z*. For example, a draughtsman thought to improve upon the designer's joint and cut the cover length from *VZ* down to *VX* on the working drawings. The mistake was happily discovered in the yard just prior to shipment, and the rivets were cut out and the joint made good. The cost was rather large, as erection was delayed at the site.

Fig. 76 is a modified form of the foregoing flange splice.

Freight charges are made on either the weight or the overall cubic capacity of the material shipped, whichever is more favourable to the shipowner. A girder which is being sent by sea in small lengths would never be given the joint of Fig. 74 with its projecting plates, because the unriveted and comparatively flexible flange plates would get badly distorted. For these two reasons projecting ends of flange plates are never permitted when the girder is being sent abroad in suitable sizes.

The inner plate is jointed at the centre line of the detail marked *C.L.*, and the other flange plates are stepped back from this line so that no piece projects past the joint of the inner plate. The web plate and main angles are also broken at this point for shipment. The two pieces, coloured solid black, are sent away bolted tempo-

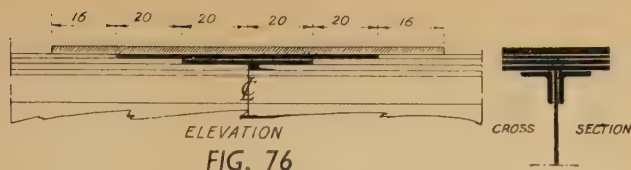


FIG. 76

rarily to the cover, which is hatched on the drawing. Each step of the coloured plates is of sufficient length to contain the requisite number of rivets in single shear which will develop the plate immediately underneath it. On arrival at site the centre pieces are dropped into position and the whole riveted up.

If the black plates and hatched cover remain riveted to the right-hand portion of the girder, the left-hand portion can travel towards the left on single shearing the three groups of 16, 20 and 20 rivets on the left of the *C.L.* The main angles and web are not taken into account in providing help to the flange plates. The number of rivets in the end of the hatched cover theoretically only require to be 16 in order to develop the $\frac{7}{16}$ -in. outer flange plate: see remarks above regarding the developing of the tensile strength of the cover by means of the end rivet groups.

The cover plate thickness is that of the thickest plate ($\frac{9}{16}$ in.) plus 10 per cent., i.e., $\frac{5}{8}$ in. This cover plate is approximately one and a half times the length of that used in the previous flange plate splice.

CHAPTER VI

SPLICES FOR ANGLES, JOISTS AND CHANNELS

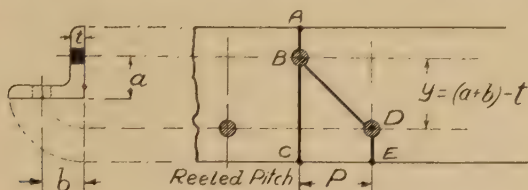
ANGLES, joists, channels and tees can act individually, either as tension or, because of their rigidity, as compression members.

Where a splice occurs in any of these sections, when acting as tension members, the force is abstracted from the section by the rivets on one side of the joint and transferred into the covers, is then carried in these covers across the joint, and ultimately finds its way back into the other portion of the section through the remaining group of rivets.

When the section carries an axial compressive load the theoretical function of the covers is to prevent lateral or side slip of the butting ends of the joint relatively to each other. The butting ends of compression members are machined to flat surfaces in order that they may bear evenly against each other, and thus the force is transferred from one part of the member to the other by direct contact of the machined ends. However, this perfect bearing of surfaces can only be attained by exercising elaborate care in workmanship, and it is therefore questionable if the ideal case is ever realised in practice.

Probably a more important point than the foregoing is the eccentricity of the load on the section. There is generally, however unintentional, an eccentricity of loading which causes bending with consequent additional fibre stresses. Eccentricity of loading is more dangerous with a compression member than with a tension member, because in the former case an increase in the load is accompanied by an increased buckling or displacement of the centre line of the section away from the load line, while in the latter case the centre line exhibits a greater tendency to coincide with the load line. Fig. 135, of Chapter VIII., shows the load P applied longitudinally to an angle through the rivets on the rivet line. The eccentricity of the line of action of P from the centre of gravity line of the angle is e_1 , and the resulting bending moment is Pe_1 . This "unintentional" eccentricity is small, and its effects are always neglected in the design calculations. If the eccentricity is larger than that due to the rivet lines then special provision should be made in the covers and the rivets of the splice to carry both the direct load and the bending

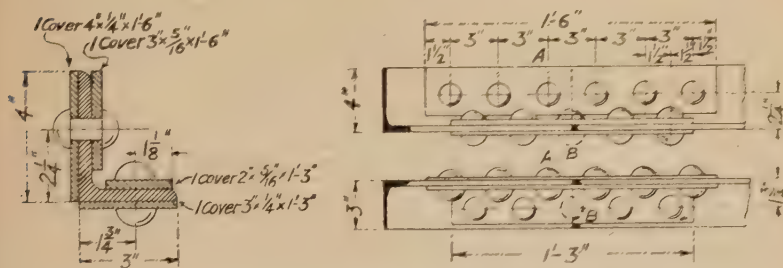
stresses created by the eccentricity, i.e., if the load and not the member is to be developed by the covers and their rivets. However, complete safety can be obtained by making the splice covers and rivets of both tension and compression members develop the full net tensile strength of the section, even though this strength is much larger than the actual load carried. Therefore, no matter



whether machined ends butt or whether the load is axial or not, the strength of the joint is the strength of the original member adopted from the calculations.

To Splice a $4'' \times 3'' \times \frac{3}{8}''$ Angle. The tensile load need not necessarily be known.

As previously stated, the various possibilities of zigzag tearing are not fully known, and as an indication of the different require-



SPLICE OF A $4 \times 3 \times \frac{3}{8}$ ANGLE

FIG. 77

RIVET HOLES $\frac{3}{4}$ DIA.

ments compare the following specification with that of the B.S.S. mentioned on the second page of Chapter V. : "In calculating the net area of any tension member the greatest number of rivets that can be cut by a transverse plane perpendicular to the axis of the member, or come within 1" (others state $1\frac{1}{2}''$ to 4") of the plane, is to be deducted from the gross transverse area." By this specification, using the 3 in. pitch or $1\frac{1}{2}$ in. reeled pitch of Fig. 77, the net area of the angle to be developed is the net transverse area of

2.48 sq. in. gross — one hole $\frac{3}{8}'' \times \frac{3}{4}'' = 2.20$ sq. in. net. By the B.S.S. only straight through or transverse tearing would occur if the reeled pitch were increased to 3.84 in. See Table 11, Appendix, "Sectional Areas of Angles with Single Rivet Lines," $y = 3\frac{5}{8}$ in.

The diagonal tearing of an angle is more easily considered if the angle is developed out into the equivalent flat. Thus in the key diagram to Table 11, Appendix, but also given on p. 75, the $4'' \times 3'' \times \frac{3}{8}''$ angle would be equivalent to a rectangle $6\frac{5}{8}''$ wide (not $7''$) $\times \frac{3}{8}''$ thick. The two rivet lines would still be at the same distances from the edges, but would be $\frac{3}{8}$ in. nearer to each other. AB (of Table 11) $= 1\frac{3}{4}''$, $y = 2\frac{1}{4}'' + 1\frac{3}{4}'' - \frac{3}{8}'' = 3\frac{5}{8}''$ and $DE = 1\frac{1}{4}''$. With $P = 1\frac{1}{2}''$ the gross length of path $ABDE = 1\frac{3}{4}'' + 3.923'' + 1\frac{1}{4}''$, while the net length in accordance with the B.S.S. is $(1\frac{3}{4}'' - \frac{3}{8}'') + (3.923'' - \frac{3}{4}'')\frac{4}{5} + (1\frac{1}{4}'' - \frac{3}{8}'') = 4.79''$; thus giving a net zigzag area of $4.79'' \times \frac{3}{8}''$ thick $= 1.80$ sq. in. net.

Gross cross-sectional area

$$\text{of angle} = (4'' + 3'' - \frac{3}{8}'') \times \frac{3}{8}'' = \text{sq. in. } 2.48$$

Net transverse area of

$$\text{angle} = 2.48 - 1 \text{ hole } \frac{3}{4}'' \times \frac{3}{8}'' = \text{,, } 2.20$$

Net diagonal area of angle = as before

$$= \text{,, } 1.80$$

Tensile strength of angle = net area $\times F_t$

$$= \text{tons } 1.80F_t$$

$$\text{Value of 1 rivet in } D.S. = F_{ds} \times \text{area} = \frac{4}{3}F_t \times 0.442$$

$$= \text{,, } 0.59F_t$$

$$\text{Value of 1 rivet in } \frac{3}{8}'' B = F_b \times \text{area} = \frac{4}{3}F_t \times \frac{3}{8}'' \times \frac{3}{4}''$$

$$= \text{,, } 0.38F_t$$

Number of rivets to

$$\text{develop angle} = 1.80F_t \div 0.38F_t = 5$$

United width of angle legs $= 4'' + 3''$

$$= \text{in. } 7$$

$\therefore$ Number of rivets in

$$3'' \text{ leg} = \frac{3}{7} \text{ of } 5 = 2$$

$\therefore$ Number of rivets in

$$4'' \text{ leg} = \frac{4}{7} \text{ of } 5 = 3$$

Covers. Total net area req. $= 1.80 + 5$ per cent. $= \text{sq. in. } 1.89$

(The cover areas are symmetrically placed, hence only 5 per cent. excess.)

$$\text{Amount to } 3'' \text{ leg} = \frac{3}{7} \text{ of } 1.89 = \text{,, } 0.81$$

$$\text{Amount to } 4'' \text{ leg} = \frac{4}{7} \text{ of } 1.89 = \text{,, } 1.08$$

Given:— $3''$ leg 1 flat $3'' \times \frac{1}{4}'' + 1$ flat $2'' \times \frac{5}{16}''$

Gross area $= 1.37$ sq. in. total. Total net

$$\text{area} = \text{,, } 0.95$$

$4''$ leg. 1 flat $4'' \times \frac{1}{4}'' + 1$ flat $3'' \times \frac{5}{16}''$

Gross area $= 1.94$ sq. in. total. Total net

$$\text{area} = \text{,, } 1.52$$

It is desirable that the elongation of every cover should be the same, otherwise a larger load will be given to the more resisting plate with a consequent eccentricity of loading at the joint. For this reason, subject to practical sizes, the covers have been given the same area on each side of any one leg, and the total metal in the covers is proportional to the amount of metal in the leg which they cover.

Checking the Joint.

1. Rivets on one side of the joint failing by crushing, 5 rivets @ $0.38F_t$; i.e., 3 in 4" leg and 2 in 3" leg = tons $1.90F_t$
 2. Main angle failing by zigzag tearing. Strength was = „ $1.80F_t$
 3. The covers on the 4" leg tearing at *A* and those on the 3" leg at *B*. Strength = net area $\times F_t = 1.52F_t + 0.95F_t =$ „ $2.47F_t$
- Efficiency in tension = $1.80F_t \div 2.48F_t = 0.73$ or = 73%

Rivet Pitch. If the design is governed by the B.S.S., then any increase of the rivet pitch will give a corresponding increase of angle strength due to the longer diagonal path for tearing. The maximum pitch, however, cannot exceed $16t = 4"$.

The 2-in. wide cover should be at least $2\frac{1}{4}$ in. wide to agree with the rule "that the least distance from the centre of a rivet hole to the edge of the plate should be $1\frac{1}{2}d$." An attempt could be made to fulfil this condition by adopting a cover wider than 2 in., and grinding down one edge to fit into the root fillet of the angle, or, alternatively, by using an interior $\frac{5}{16}$ -in. thick bent plate to cover both the 4-in. and the 3-in. legs of the original section.

The rivet shown dotted is a stitching rivet holding the covers close against the main angles, but it adds nothing to the strength of the joint. It is required because of the rule $16t = 16 \times \frac{1}{4} = 4"$ maximum pitch for tension members and $12t = 3"$ for compression members. The angles and covers are assembled, bolted together, and the hole then drilled through the covers and main angles.

When the $4" \times 3" \times \frac{3}{8}"$ Angle is a Strut. The above joint is equally applicable when the angle acts as a strut. In fact, it would be overstrong if the load were axially applied. The load, however, is always applied to the angle by the rivets on the rivet line, and, therefore, the angle is non-axially loaded. Thus there is a bending moment, already spoken of as "unintentional," existing in both the tension and compression splices.

In the case of a compression member the thrust coming on the edge of a rivet-hole is transferred across that hole by the rivet

shank, which is assumed to fill the hole completely. This does not apply to a tension member where the load is a pull. Net areas are, therefore, used in tension members, but gross areas for compression members.

$$\begin{aligned}\text{Effective strength} &= \text{effective area} \times \text{permissible stress}, \\ &= \text{gross area} \times F_c = 2.48F_c.\end{aligned}$$

Now F_c the working compressive stress is generally less than F_t the corresponding working tensile stress. The longer the angle strut the smaller is the working stress, but assuming an l/k about 100, *i.e.*, the angle is in the region of 5 ft. long, the working compressive stress F_c will be about $0.5F_t$, depending upon end conditions and formula used (see Chapter III., Vol. II.).

$\therefore$ Number of rivets to

$$\begin{aligned}\text{develop gross area of angle} &= 2.48F_c \div 0.38F_t \text{ (i.e., bearing)} \\ &= 2.48 \times 0.5F_t \div 0.38F_t = 4 \text{ rivets}\end{aligned}$$

$$\text{Gross area of covers required} = 2.48 + 5\% = 2.60 \text{ sq. in.}$$

which results, of course, disregard the eccentricity of the load, unintentional or otherwise. The tension splice gives 5 rivets and 3.31 sq. in. gross of cover area and thus makes allowance for any eccentricity of loading, etc., because the fallacy in the foregoing calculation is that F_c is purposely reduced in value as the strut becomes longer thereby making an allowance for the inherent bending stresses in a column.

Splices for Joists, Channels, etc. Whether these sections act as tension members, compression members, or as beams, it will be found, if the net tensile strength of a section is developed by the splice covers and their rivets, that the spliced member can act in any of these capacities. Further, it often happens that a design drawing indicates a splice, but neither the stresses nor the loads acting are given, so that the only satisfactory splice is that which develops the net tensile strength of the member.

As an example, the splice for an $8'' \times 6'' \times 35$ lb. R.S.J., Fig. 188, will be given here from the point of view of the development of the net tensile value, while in Chapter X. the same splice will be examined from the beam aspect by using equivalent moments of inertia, etc.

Gross area of joist		= sq. in. 10.3
Gross area of web	= $8'' \times 0.35''$	= „ 2.8
Gross area of one flange	= $\frac{1}{2}(10.3 - 2.8)$	= „ 3.75

Areas required.

Web covers, gross	= $2.8 + 5\%$	= „ 2.94
Web covers, net	= $(2.8 - 2 \text{ holes } @ \frac{3}{4}'' \times 0.35'') + 5\%$	= „ 2.38

Flange cover, gross	= $3.75 + 10\%$	= sq. in. 4.13
Flange cover, net	= $(3.75 - 2 \text{ holes } @ \frac{3}{4}" \times 0.648") + 10\%$	= ,, 3.06

Areas given.

Web covers	2 @ $5\frac{1}{4}" \times \frac{5}{16}"$, see Table 5, Appendix; gross area = ,, 3.28
	net area = $3.28 - 4 @ \frac{3}{4}" \times \frac{5}{16}"$ = ,, 2.34
Flange cover	1 @ $6" \times \frac{1}{16}"$ per flange; gross area = ,, 4.13
	net area = $4.13 - 2 @ \frac{3}{4}" \times \frac{1}{16}"$ = ,, 3.10

Rivets, $\frac{3}{4}"$ diameter.

Value per rivet in terms of F_t , which need not be known.

Single Shear (<i>S.S.</i>)	= $0.442 \times \frac{2}{3}F_t$	= tons 0.3 F_t
Double Shear (<i>D.S.</i>)	= $0.442 \times \frac{4}{3}F_t$	= ,, 0.59 F_t
Bearing, 0.35" web	= $0.35 \times \frac{3}{4} \times \frac{4}{3}F_t$	= ,, 0.35 F_t
Number req'd to de- velop net web covers (<i>D.S.</i>)	= $2.34F_t \div 0.35F_t$, <i>B.</i> (and <i>D.S.</i>)	= 6
Number req'd to de- velop net flange cover (and <i>B.</i>)	= $3.10F_t \div 0.3F_t$, <i>S.S.</i>	= 10

Checking the Joint.

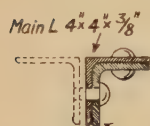
Net strength of joist; = (10.3 - 4 @ $\frac{3}{4}"$ tearing at <i>CC</i> $\times 0.648") F_t$	= tons 8.36 F_t
Net cover strength. All four tearing at <i>AA</i>	= $(2.34 + 2 @ 3.10) F_t$ = ,, 8.54 F_t
Rivet strength; all = web, 6 @ 0.35 F_t + flanges, rivets failing 20 @ 0.3 F_t	= ,, 8.1 F_t
The rivet value 8.1 F_t is 3 per cent. below 8.36 F_t , but accept.	

Non-Symmetrical Angle Splice, Figs. 78 and 79. The angle is $4" \times 4" \times \frac{3}{8}"$ with $\frac{3}{4}$ -in. diameter rivets.

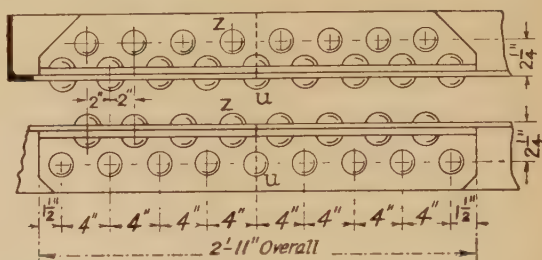
As the first of these figures shows (in heavy line and hatched cross-section) the rivets are in single shear and $\frac{3}{8}$ -in. bearing. This joint is really an eccentric one with a bending moment acting in addition to the direct load, the value of the moment being $P \times e$, where P is the force in the bar and e is the eccentricity of the centre of gravity of the cover from the line of action of P . The load travels along the main angle from the right, and, near the joint, side-tracks through the rivets into the cover. It then travels the length of the cover which bridges the gap or joint, and finally returns to the left-hand portion of the main angle through the second group of rivets. To make allowance for this eccentricity of loading

the B.S.S. specifies that the cover shall be 10 per cent. in excess of the area covered. In the example the rivets which connect the cover

RIVET HOLES $\frac{3}{4}$ " Dia



Cover $3\frac{5}{8} \times 3\frac{5}{8} \times \frac{1}{2}$ " $\times 2'-11"$
out of a $4 \times 4 \times \frac{1}{2}$ " L
or alternatively a bent plate.



NON SYMMETRICAL JOINT OF A $4 \times 4 \times \frac{3}{8}$ " ANGLE

FIG. 78

FIG. 79

to the main angle will be given in sufficient number to develop this extra 10 per cent. strength of the covers.

The main angle may occur in the flange of a plate girder as indicated by the dotted portion of the cross-section. Due to the proximity of the web plate and flange plates, double covers, such as were used in the preceding case, cannot be employed. Further, no help is assumed to be given to the jointed angle by the adjacent elements of the structure other than the cover.

Occasionally round-backed angles are specified as covers. These angles are seldom rolled, and it is generally cheaper both in time and money to specify an ordinary angle with the heel machined to fit into the root of the main angle. Alternatively, a flat plate may be bent to suit.

The joint will be designed on the assumption that the main angle is in tension, as explained in the previous examples.

Gross cross-sectional

area of $4" \times 4" \times \frac{3}{8}"$

$$\text{angle} = (4" + 4" - \frac{3}{8}") \times \frac{3}{8}" = \text{sq. in. } 2.86$$

Net transverse area of

$$\text{angle} = 2.86 \text{ gross} - 1 \text{ hole } \frac{3}{4}" \times \frac{3}{8}" = \text{,, } 2.58$$

Net diagonal area of

$$\text{angle as explained under} = \text{,, } 2.18$$

Tensile strength of = net area $\times$ working

$$\text{angle} \quad \text{stress } F_t = \text{tons } 2.18 F_t$$

Value of 1 rivet in S.S. = $F_s \times 0.442 = \frac{2}{3} F_t \times 0.442$

$$(\frac{3}{4}" \text{ dia.}) = \text{,, } 0.3 F_t$$

Value of 1 rivet in $\frac{3}{8}" B = F_b \times \frac{3}{8}" \times \frac{3}{4}"$

$$= \frac{4}{3} F_t \times \frac{9}{32} = \text{,, } 0.38 F_t$$

Cover, net area required = 2.18 sq. in. + 10% = sq. in. 2.40

Rivets required to de-

velop the cover $= 2.40F_t \div 0.3F_t = 8$

Net Diagonal Tearing Area. Refer to the key diagram of Table 11, p. 75. Here $P = 2''$ and $AB = DE = 1\frac{3}{4}''$, while $y = 2\frac{1}{4}'' + 2\frac{1}{4}'' - \frac{3}{8}'' = 4\frac{1}{8}''$. The gross length of BD is thus 4.584". The net zigzag path for tearing of main angle is $(1\frac{3}{4}'' - \frac{3}{8}'') + (4.584'' - \frac{3}{4}'') \frac{4}{5} + (1\frac{3}{4}'' - \frac{3}{8}'') = 5.817''$.

Whence the net zigzag area is $5.817'' \times \frac{3}{8}''$ thick = sq. in. 2.18.

Given :—1 Bent plate or machined angle $3\frac{5}{8}'' \times 3\frac{5}{8}'' \times \frac{1}{2}''$.

Gross cross-sectional

area of cover $= (3\frac{5}{8}'' + 3\frac{5}{8}'' - \frac{1}{2}'') \times \frac{1}{2}'' =$ sq. in. 3.38

Net transverse area

of cover $= 3.38 - 1 \text{ hole } \frac{3}{4}'' \times \frac{1}{2}'' =$ „ 3.00

Net diagonal area of cover as explained for

main angle $=$ „ 2.60

Rivets on each side of joint, per leg

$=$ „ 4

Checking the Joint.

1. The rivets on one side of the joint failing by single shear, the lesser rivet value, *i.e.*,

8 rivets @ $0.3F_t =$ tons 2.4 F_t

2. Main angle failing by zigzag tearing. Strength

was $=$ „ 2.18 F_t

3. The bent cover plate tearing diagonally at

ZU, Fig. 79 $=$ „ 2.60 F_t

Joint efficiency in tension $= 2.18F_t \div 2.86F_t = 0.76$

or 76%

Riveting. Note that the cover encroaches on the clearance for the rivet heads in the adjoining leg. Because of this the thicker the cover the smaller is the permissible rivet diameter. Alternatively, the “standard” spacing of the rivet lines, *i.e.*, from heel to rivet line, could be departed from in the length of the splice.

Grouped and Reeled Splice, Figs. 80 to 83. This form of splice is often used for splicing the main angles of plate girders. The practice in this country is to limit the length of the angles to about 40 ft. It may be valuable on certain occasions to have these angles over 40 ft. in length, but when over this length work is not facilitated in the shop. The practice in America is to specify that no splices will be permitted for the flange angles of plate girders if these can be obtained of the required length from the mills. Anent this, one American mill quotes “maximum length for all angles 100 feet.”

No help is supposed to be given to the joint covers by the web or

flange plates as these are working, presumably, at their full capacity. The joint has thus been shown without web or flange plates to avoid confusing the reader. American practice seldom makes use of this type of joint, preferring the simpler one discussed in the previous article.

The angles are $6'' \times 6'' \times \frac{1}{2}''$, and each angle therefore has four rivet lines. The design of the girder fixed the diameter of the rivet hole at $\frac{15}{16}''$. Design from the point of view of tension.

Rivet Values.

$$\text{Single shear} = 0.7854 \left(\frac{15}{16}\right)^2 \times \frac{2}{3} F_t \text{ tons per rivet} = 0.46 F_t$$

$$\text{Bearing } \frac{1}{2}'' \text{ Pl.} = \frac{15}{16}'' \times \frac{1}{2}'' \times \frac{4}{3} F_t \text{ ,, ,, ,,} = 0.63 F_t$$

$$\text{Bearing } \frac{7}{16}'' \text{ Pl.} = \frac{15}{16}'' \times \frac{7}{16}'' \times \frac{4}{3} F_t \text{ ,, ,, ,,} = 0.55 F_t$$

Rivet failure will occur due to single shear.

Net Area of One Main Angle

$$= 5.75 \text{ sq. in.} - 2 \left(\frac{1}{2}'' \times \frac{15}{16}''\right) = 4.81 \text{ sq. in.,}$$

since at any cross-section of the angle only two holes occur.

Let V_1 = the total strength of the group of rivets in one plane between the joint and the end of one angle.

V_2 = ditto in leg of angle perpendicular to the foregoing leg.

V_3 = the same as for V_1 , but in the other main angle.

V_4 to V_9 inclusive are then distributed in a similar manner.

c = the net area of each cover in sq. in.

a = the net area of each main angle = as above, 4.81 sq. in.

Case 1. Failure by single shear should equal the strength of the main angles, Fig. 82.

$$\begin{array}{ccccccc} (V_1 + V_2) & + & V_5 & + & (V_8 + V_9) & = & 2aF_t \quad . \quad . \quad 1 \\ \text{(Cover to main} & & \text{(Main angle to} & & \text{(Cover to main} & = & \text{original strength of} \\ \text{angle.)} & & \text{main angle.)} & & \text{angle.)} & & \text{main angles.} \end{array}$$

$$\begin{array}{lcl} \text{If all the } V\text{'s are equal, then } 5V & = & 2aF_t \\ & \text{whence } V & = \frac{2}{5}aF_t \quad . \quad . \quad 2 \end{array}$$

Case 2. Both angle covers tearing and V_5 failing in single shear, Fig. 83.

$$cF_t + V_5 \text{ main angle to main angle} + cF_t = 2aF_t \quad . \quad . \quad 3$$

$$\therefore cF_t = aF_t - \frac{1}{2}V \quad . \quad . \quad 4$$

since $V_5 = V$, if the V 's are the same.

Case 3. Tearing straight through at a joint of the main angle as in Fig. 81. The metal torn will be on the line of rivets adjacent to the joint, and the tear will be through the remaining main angle and the two covers.

$$aF_t + cF_t + cF_t = 2aF_t \quad . \quad . \quad 5$$

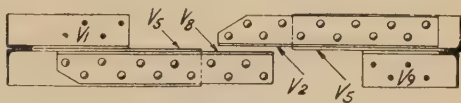
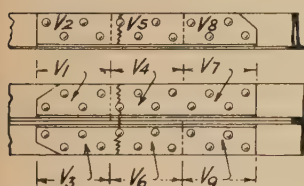
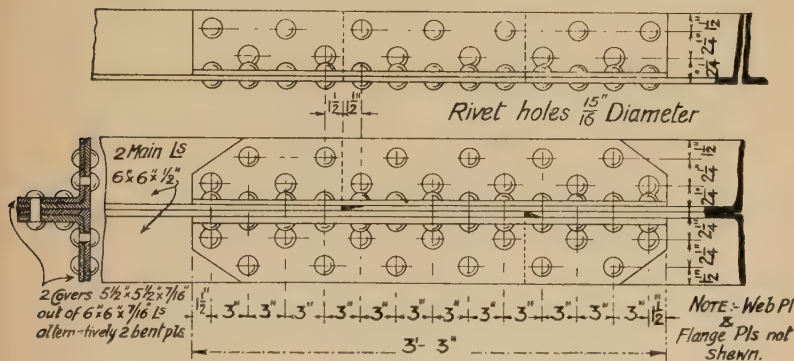
$$\text{whence } c = \frac{a}{2} \quad . \quad . \quad 6$$

From 2. $V = \frac{2}{5} \times 4.81F_t = 1.92F_t$ and the
 req'd No. of rivets $= 1.92F_t \div$
 $0.46F_t = 4.2$, say 4 7

From 4 and 7. $cF_t = 4.81F_t - \frac{4 \times 0.46F_t}{2}$
 whence $c = 3.89$ sq. in. 8

From 6. $c = 2.4$,, 9

Two values are obtained for c because that of equation 4 depends



partly upon the number of rivets used, while that of equation 6 depends entirely upon the area of the main angles.

The joint, considered as a whole, is symmetrical. Equation 9 gives the net cover area required, as it is independent of rivet values. Area required is 2.4 sq. in. $+ 5\% = 2.52$ sq. in.

Covers Adopted. $5\frac{1}{2}'' \times 5\frac{1}{2}'' \times \frac{7}{16}''$. Net area of one $= 4.62 - 2$ holes $\frac{1}{16}'' \times \frac{7}{16}'' = 3.8$ sq. in. The value for c in equation 8 is 3.89 sq. in. if 4 rivets per group are used. If the number of rivets per group be altered, 3.89 alters. The covers adopted thus satisfy both 8 and 9, provided that at least 4 rivets per group are used.

Checking the Joint. Substitute the values in the left-hand side of the following equations :—

In equation 1. 5 groups of 4 rivets @ $0.46F_t$ per rivet

$$\text{(Fig. 82)} \quad = 9.20F_t$$

In equation 3. $3.8F_t + 4 @ 0.46F_t + 3.8F_t$ (Fig. 83) = $9.44F_t$

In equation 5. $4.81F_t + 3.8F_t + 3.8F_t$ (Fig. 81) = $12.41F_t$

Net strength of two main angles = $4.81F_t \times 2$ = $9.62F_t$

Since only 4, and not 4.2, rivets of 7 can be used, the riveting is slightly under strength. By increasing groups V_4 , V_5 and V_6 of Fig. 81 by one rivet each ($0.46F_t$) the symmetrical joint of Fig. 80 is obtained with increased strength, since :—

$$1 \text{ is now } 9.20F_t + 0.46F_t = 9.66F_t$$

$$3 \text{ becomes } 9.44F_t + 0.46F_t = 9.90F_t$$

The covers do not encroach too much upon the "fairway" on the inner side of the inner rivet line, and clearance is thus left for machine riveting. Had the cover been thicker, sufficient clearance between the inner rivet line and the side of the cover might not have been obtained. The standard rivet spacing, reading from the heel outwards, Fig. 80, is $2\frac{1}{2}$ in., $2\frac{1}{2}$ in. and $1\frac{1}{2}$ in., and this would hold good on both sides of the splice. At the splice the spacing could be altered to $2\frac{1}{2}$ in., 2 in. and $1\frac{1}{2}$ in., thus giving an extra $\frac{1}{2}$ in. between the inner rivet line and the heel, to accommodate the thick cover.

Minimum Rolled Pitch. From what has been said in Chapter 1, upon the cross-sectional area of angles, that of the $6' \times 6' \times \frac{1}{2}'$ angle of Fig. 68 is equivalent to a rectangle $11\frac{1}{2}"$ wide (not $12"$) $\times \frac{1}{2}"$ thick. In other words, the angle may be regarded as having been flattened out or developed. The two inner rivet lines will still be at the same distance from the toes of the angle, but will be 2 in. from the heel or centre line instead of $2\frac{1}{2}$ in.

If the angle fails by tearing it has four very probable paths :—
(a) Straight across along the line $ABDG$; (b) zigzag path $ABCEF$; (c) zigzag $ABCDEG$; and (d) $ABCDG$. The distances, centre to centre of holes, are given on the diagram.

The B.S.S. specifies that only four-fifths of the *net* diagonal distances shall be taken in computing the effective area. The reason for this reduction factor was given in the previous chapter under the heading of Zigzag Tearing.

$$(a) \text{ } ABDG. \text{ Effective length} = 11.5' - 2 @ \frac{13}{16}' = 9.625'$$

$$(b) \text{ } ABCEF. \text{ Effective length} = (1.5' - \frac{13}{16}') + \frac{4}{5} (3.75' - \frac{13}{16}') + 7.75' - 1\frac{1}{2} \times \frac{13}{16}' = 9.625'$$

$$(c) \text{ } ABCDEG. \text{ Effective length} = 2 (1.5' - \frac{13}{16}') + 2 \times \frac{4}{5} (3.75' - \frac{13}{16}') + \frac{4}{5} (5' - \frac{13}{16}') = 9.8125'$$

$$(d) \text{ } ABCDG. \text{ Effective length} = (1.5' - \frac{13}{16}') + \frac{4}{5} (3.75' - \frac{13}{16}') + \frac{4}{5} (5' - \frac{13}{16}') + (3.75' - \frac{13}{16}') = 9.8125'$$

The tearing value of the main angles assumed in the calculations was the transverse area of $3.625" \div \frac{1}{2}" = 4.81$ sq. in. and agrees with (a) and (b). The sealed patch of 3 in. is the minimum sealed patch for transverse tearing to occur. Had the holes been 1 in. diameter the respective lengths would be 9.5 in., 9.45 in., 9.4 in. and 9.4 in., i.e., diagonal tearing along *ABCEP* giving a net area of $9.45" \div \frac{1}{2}" = 4.725"$. Similarly, had the sealed patch been $2\frac{1}{2}$ in. instead of 3 in. for the $\frac{1}{2}$ in. diameter holes, the effective diagonal area would have been less than the transverse area.

Compare these results with the net areas as given in Table 12, Appendix, which has the key diagram lettered to correspond with the foregoing text.

Diagonal Tearing of the Cover. The width of the $5\frac{1}{2}" \div 5\frac{1}{2}" \div \frac{1}{8}"$ cover angle when developed out is $2 \div 5\frac{1}{2}" = \frac{1}{8}" = 10\frac{1}{8}"$.

(a) *ABIDG*—Effective length = $10\frac{1}{8}" - 2 \times \frac{1}{8}" = 9.69"$

(b) *ABCEP*—Effective length = $(1.5" - \frac{1}{8}") - \frac{1}{2}(3.75" - \frac{1}{8}") = 6\frac{1}{8}" - 1\frac{1}{2} \times \frac{1}{8}" = 4.69"$

(c) *ABCEDEP*—Effective length = $2(1.5" - \frac{1}{8}") + 2 \div \frac{1}{2}(3.75" - \frac{1}{8}") + \frac{1}{2}(4.25" - \frac{1}{8}") = 9.21"$

(d) *ABIDG*—Effective length = $(1.5" - \frac{1}{8}") + \frac{1}{2}(3.75" - \frac{1}{8}") + \frac{1}{2}(4.25" - \frac{1}{8}") + (3.75" - \frac{1}{8}") = 9.21"$

The tearing value of the cover angle was assumed to be the transverse area of $8.45" \div \frac{1}{8}" = 3.8$ sq. in., which is in keeping with the above calculations.

OTHER TYPES OF FLANGE ANGLE SPLICES

As pointed out when dealing with the angle splice of Fig. 30, it often happens that the cover angle required is so thick that the rivet heads cannot be closed with standard machine dies. Dies are sometimes ground down so that clearance is obtained, but this may result in a weak rivet head. Another alternative is to depart from the standard rivet head for the length of the joint and make the distance, heel of angle to centre of rivet, greater than the standard suggests.

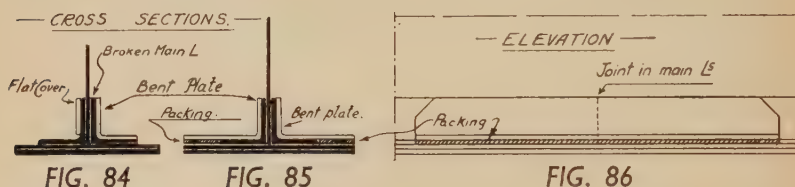
It often arises, especially with small-legged thick angles and angles with double rivet lines, that a change of position of the rivet line brings the rivets too close to the edge of the section. When this occurs Fig. 34 shows a method of lowering the bent cover-plate thickness by placing part of the cover area in the form of a flat bar next to the unbroken main angle.

Only one angle is being joined at this particular part, viz., that under the bent plate. The total area of the covers is the sum of the vertical flat and the bent plate cross-sectional areas. Where the

thicknesses are the same, the ratio of the gross areas of metal of the two covers is approximately 1 : 2 respectively. The thickness of the vertical plate rarely exceeds that of the bent plate and is usually less.

The rivets through the horizontal leg are, in accordance with the premise, in single shear and bearing, while those through the vertical leg—there being two vertical covers—are in double shear and bearing.

The broken main angle is unsymmetrically covered. It is also unsymmetrically developed by the rivets, since the vertical rivets are in double shear and those through the horizontal leg are in single



shear. Extending the covers to include an extra rivet or two would therefore not be amiss.

Figs. 85 and 86 show a type of joint which is used where a straight through break, of the two main angles, occurs. It can also be used when one angle only is broken, or it can be used in the grouped and reeled angle splice of Fig. 80. The bent plates are carried right out to the edge of the flange plates and are riveted to them through the packings. These latter rivets should not be calculated as carrying any load from the main angles, but be merely looked upon as stitching or tacking rivets. The effective rivets are those through the main angles.

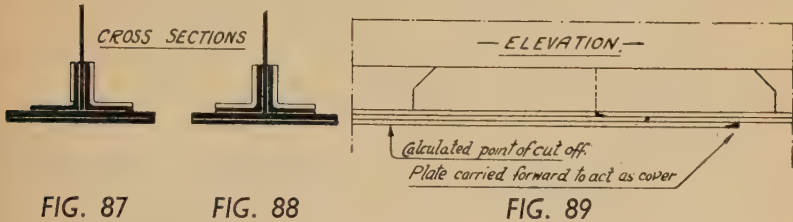
This joint, with kneed and plated stiffeners placed at or near the break of the main angles, gives lateral stiffness. It is, therefore, a good joint to use in the case of crane girders, whose flanges have often to act as horizontal girders in resisting the dragging of weights across the shop floor in a direction at right angles to the crane girder. Crane girder compression flanges are generally unsupported extraneously in a lateral direction, and any extra strength and rigidity at a flange angle joint is therefore worthy of consideration.

Just as the rivets through the packings should not be considered to take any load, so also with a portion of the metal of the bent covers. Perhaps a fair assumption would be to say that the effective cover metal is that in direct contact with the flange main angles plus one-half of that in contact with the packings.

The reason is similar to that given in Chapter VIII. relative to the

net effective area of a tension angle with an unriveted leg, as illustrated in Fig. 136.

Figs. 87 and 88, representing one main angle broken and two main angles broken, respectively, are alternative cross-sections to



the elevation of Fig. 89. The outer flange plate is made to act as an indirect flange angle cover by extending it past its point of cut-off. The horizontal legs of the flange angles are now covered top and bottom, and the rivets through them are in double shear, as well as the rivets through the vertical legs.

The bent plate covers of Fig. 85 can also be used with the extended or prolonged flange plate.

If the main angles can be broken near the point of cut-off of the flange plate this joint should be used, as it lessens the thickness of the bent plate covers. On the other hand, there is often nothing (except appearance) to hinder the designer from placing the main angle joints where he pleases, and using an additional outer cover plate on the flange. This small "flange" plate would only run the length of the joint, but in exposed work of some types would appear rather unsightly.

Rivets in Joints doing Double Duty. Sometimes it is asked, "Why are rivets which are used to transfer the shear from the web plate into the main flange angles, and which are supposedly already working at their full capacity, also called upon to carry an extra load due to an angle splice?" The answer is that the rivets are not in single or double shear, but may be in treble or even quadruple shear.

Fig. 90 shows a rivet which, carrying a load of 12 tons, is apparently dangerously overloaded. On investigating the shearing



forces it is found, however, that the single shear at any one section is well within the permissible value.

1" dia. rivet. Single-shear value $= 0.7854 \times 6^2$
 with $F_t = 9\pi/\text{sq. in.}$ $= 4.71$ tons

At the common face of plates a and d the shear is $= -2$ „

At the common face of plates d and b the shear is
 $-2 + 6$ $= +4$ „

At the common face of plates b and e the shear is
 $-2 + 6 - 8$ $= -4$ „

At the common face of plates e and c the shear is
 $-2 + 6 - 8 + 6$ $= +2$ „

The maximum single-shear value of 4 tons is under the permissible value of 4.7 tons. When cover plates are riveted to the outside of main angles the extra load brought on to the rivet is applied, as a single shear, on the common surface between the added cover and the outer face of the main angle, and the permissible shearing value is not exceeded.

Bearing Value. The pitch of the rivets through the flange main angles would be, in all probability, settled by the bearing value of the rivets on the web plate.

As a rule the web plate is thinner than either of the main angles, and certainly thinner than their added thicknesses.

If, therefore, the bearing value of the rivet is safe with the web, it is still more safe on the main angles, and safer still at the main angle covers of a joint, since these are always at least 5 per cent. thicker than the main angles.

CHAPTER VII

SPLICES FOR THE WEB PLATES OF PLATE GIRDERS

Length and Thickness of Web Plates. The depth of the web plate of a plate girder is generally about four times the width of the flange plates, and, therefore, web plates cannot be obtained in such long lengths from the mills. Even if they could be obtained they would prove to be too heavy and awkward to handle in the shops. The web plate, if over 30 ft. or thereby in length, is usually built up of two or more lengths butted together and the joint covered with a splice plate on each face of the web. Further, the presence of a splice permits of a camber being given to the girder which would otherwise be difficult to obtain if the plate were in one piece (see the article on Camber, Chapter I., Vol. II.).

In heavy girders of large span it is economical to vary the web thickness twice or thrice, and the web splices permit of this alteration of section.

The following table may be of service to the beginner, but it must be understood that it is only a rough guide. The number of joints, or splices, will be one less in number than the number of pieces forming the length of the web plate.

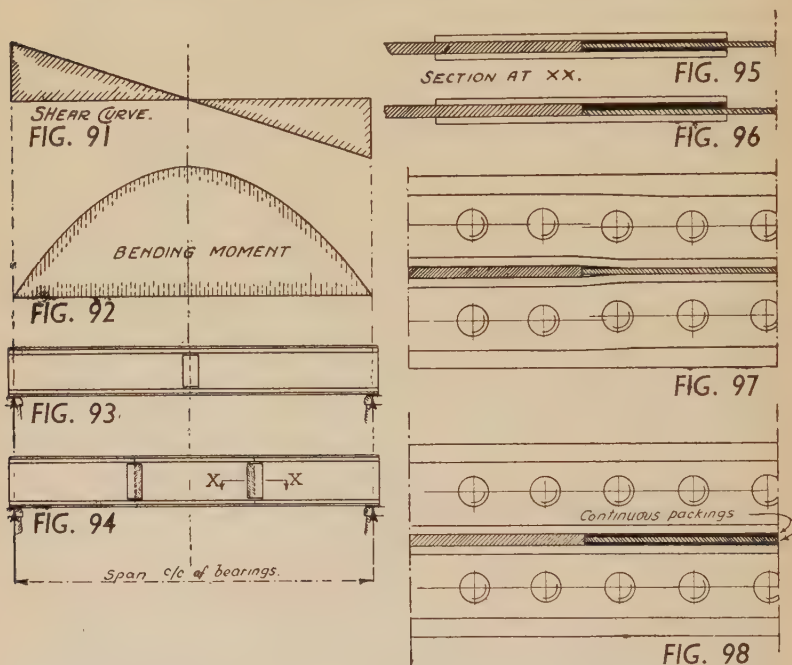
Overall Length of Girder in Feet.	Thickness of Web Plate.	Number of Pieces forming Web.
About 30—35	Same throughout	One (or two).
40	" "	Two.
50	" "	Two (or three).
60	" " if light	Two, three or four.
60	Two different thicknesses if heavy .	Three or four.
70	Same throughout if light	Three or four.
70	Two (or three) different thicknesses if heavy	Three, four (or five)

The term "light" is used for web plates up to and including $\frac{1}{2}$ in. thick.

The term "heavy" is used for web plates including and over $\frac{3}{4}$ in. thick.

Web thicknesses between $\frac{1}{2}$ in. and $\frac{3}{4}$ in. thick would fall either into the light or heavy category, according to the design and the designer.

The position of the web splice depends upon the limiting size of the plate rolled (see Table 6, Appendix), and perhaps also upon the position of extraneous connections to the girder. In a crane girder there is practically no restriction upon the position of the web splice, and the same may be said for a deck span bridge. In a through span



bridge with cross girders the splice must not occur at the junction of the cross to the main girder. Its position is therefore fixed by symmetry midway between two cross girders. Further, splices should always be placed so that a symmetrical appearance is obtained. The reason is partly æsthetic and partly economic. The latter reason will be apparent from Fig. 94, since the outer left-hand portion is similar but "other hand," i.e., a reflection, to the outer right-hand portion. In this way templates, etc., are reduced in number and, therefore, also the cost of fabrication.

Diagrams 93 and 94 show alternative schemes for a girder which carries a uniformly distributed load. In the former diagram, since there is only one web splice, the web plate, although in two portions,

must be of the same thickness throughout. In the latter diagram the web plate is shown as being in three pieces, so that the mid-portion may be thinner than the two end pieces. This follows, of course, from the fact that the shear is much less in the mid-portion than it is in the abutment portions.

Web Packings, Figs. 95 and 96, are shown inked in and are placed under the web plate covers on the thin mid-portion of the web. These covers extend from main angles to main angles. If it is desired to have the longitudinal centre line of the girder coincident with the plane of loading and of bending, then thin packings will be required on each face of the thin portion of the web plate as illustrated in the first figure. The alternative to this method is to place a packing of twice the thickness on only one side of the thin portion of the web plate after the manner of Fig. 96. The resulting eccentricity is slight and is usually neglected. The advantage gained is that a thicker packing is obtained which is more easily worked with than the two thinner packings.

Springing of Main Angles and Packings. The main angles are often sprung from the thick portion of the web to the thin portion, Fig. 97. This is accomplished by using the high-pressure riveting machines for clenching them together, and the rivets contracting on cooling permanently exert this required force. The resulting axial tension on the rivets exists only on these rivets adjacent to the set. No special smith work is necessary unless the main angles are exceptionally heavy. This springing of angles is quite common, and another example is met with in the rafters of a roof truss. In many roof trusses the shoe and apex gusset plates are made thicker than the remaining gussets attached to the rafter bars. The rafter angles on leaving the shoe plate, therefore, come close together, so as to sandwich the next gusset plate. In this example the springing of the angles is more gentle than is the case with the plate girder main angles. The disadvantage of springing the main angles is that it upsets the rivet lines in the flange plates, as the rivets will not lie on continuous longitudinal straight lines. Conversely, if they be kept collinear in the flange plates, then at that portion where the web is thin the rivets must approach the toes of the main angles, Fig. 97.

Fig. 98 indicates how the foregoing disadvantage is removed. A continuous thin packing (the same width as the main angle leg with which it is in contact) is placed under each main angle, where these are riveted to the thin portion of the web. The main angles are now absolutely straight throughout their full length. This method is used in conjunction with that of Fig. 95.

Alternatively, the packing may be doubled in thickness and placed

on one side of the web only. Both main angles will thus again be straight throughout their full length. This latter method will be used in that girder where the web splice of Fig. 96 is adopted. Here again the plane of loading is no longer coincident with the longitudinal vertical plane of the girder. The slight eccentricity of the web plate will cause a secondary bending moment, but not of large amount.

From the foregoing remarks it will be seen that the thinning down of the web plate does not always result in a financial gain.

Comparison of Web and Flange Splices. Owing to the fact that a web splice is generally placed where the shear is not large, two methods of designing a web splice are :—

- (1) Designing for the actual shear occurring at the joint.
- (2) Designing the splice so that it is as strong as the web plate covered. The splice thus obtained is usually over strong.

Compare this with the flange splices. When the bending moment decreases less metal is required in the flanges, and one flange plate is cut off (Chapter X.). This curtailment of the flange plates is repeated until, in the case of the tension flange of a simply-supported girder, there is often only the main angles left near the abutments where the bending moment approaches zero. The same accuracy of grading the metal, according to its load, is not obtainable in the web. The angles and plates of the flanges are thus practically always worked up to their full capacity, and when these are spliced the covers must develop fully the strength of the parts covered.

British specifications limit the design of the web splice to either a combination of (1) and (2), or wholly to the second method. It is usually stated that the sectional area of the covers shall be at least 5 per cent. in excess of the area covered. If the web plate is being carried throughout at the same thickness the cover plates will be overstrong, since the web plate itself is overstrong at the joint, which is usually situated at a point of minimum shear. The rivets, however, have only to be of a sufficient number to develop the shearing force and not the full strength of the part covered.

The choice is thus left to the designer of making the rivets develop the shear, or, by using more rivets, develop the section covered. The same point was discussed with regard to the two end groups of rivets in the outer flange-plate cover of Fig. 75.

TWO METHODS OF CALCULATING WEB SPLICES. (1) When the flanges of the girder are assumed to take all the bending moment and the web is supposed to take only the shear, then the splice is designed for shear only.

(2) When the web aids the flanges in carrying bending moment, then the web splice must be designed to carry both the total shear

at the section and that proportion of the total bending moment allotted to the web.

SIMPLE WEB SPLICE FOR SHEAR ONLY

In this case the web covers and their rivets will, therefore, offer the necessary resistance to the shear; any help which the flanges may afford is disregarded. In Fig. 99 the right-hand portion of the web can only sink relatively to the left:—(a) If all the rivets through the web cover plates on one side of the joint fail by crushing (bearing) or in double shear, Fig. 101.

(b) If the web covers have their vertical cross-sectional gross area sheared through.

In Fig. 100 the metal to be cut through is $39\frac{3}{4}'' \times \frac{5}{16}'' \times 2$. Specifications state that, "The shearing stress on the web plates of plate girders shall be calculated on the gross sectional area of the full depth of the plate." This is practically equivalent to saying that the rivet shanks fill the holes in the web covers and the metal to be sheared through is the gross area and not the net area. The reasoning is the same as was used for compression members. Some designers deduct the area of the rivet holes in arriving at the effective resisting area of the web covers, but the working stress used is correspondingly higher than that of the B.S.S., which adopts, as the permissible shearing stress on gross area, seven-tenths of the working tensile stress.

It must be particularly noted that the plate web girder is not the simple structure our calculations would make it appear to be. It is extremely complex, and because of this the calculations used by the designers are really approximations, and being approximations must vary, since no clean-cut arithmetical solution can be obtained. By their specification the British Standards Institution have virtually laid down the method of calculation.

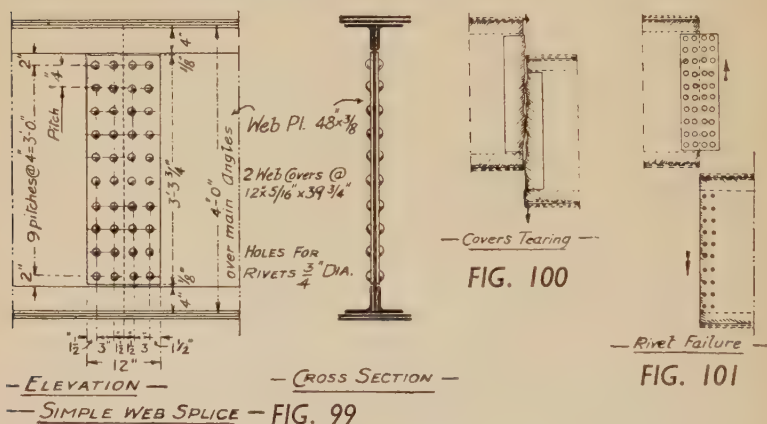
Comments on Simple Web Splices. *Always give at least two rows of rivets on each side of the joint* although the calculations may only ask for one. The vertical rivet pitch can be widened out, but must not exceed the maximum permissible pitch of twelve (or sixteen) times the thinnest plate. Two rows of rivets give a fixity or rigidity to the cover plates, and therefore to the whole joint, in a transverse direction, that is lacking where there is only one row of rivets.

In agreement with the assumption that the web takes shear only, the minimum pitch is limited to $3d$, and not to any consideration of lowering the web plate's section modulus, as happens in the case of the web which takes bending moment in addition to shear.

It is shown in Vol. II. (p. 12 and Fig. 18) that the distribution

of the shear stress on the web, *between* the top and bottom main angles, is practically uniform. The vertical pitch can thus be made uniform, and every rivet will carry approximately the same load. This is not the case with a web splice which also carries bending moment.

Students when given a uniform load on a girder often conclude that the maximum shear at a splice is that obtained from the shear diagram of Fig. 91. This is true if the load is constant and if when first applied to the girder it is distributed evenly throughout. If one end is loaded before the other this shear curve no longer holds. In this case it is equivalent to a uniform live load advancing on to



the span from either end. There must therefore be a shear, some time or other, at the girder centre line. To allow for all contingencies which may arise, it is better—whether the load is dead or live—to find the maximum possible shear at any splice by loading the girder between the splice and the more distant abutment, and to design the web splice for this shearing force.

Example of Web Splice for Shear, Fig. 99. *Given* :—Girder 4 ft. deep over angles, with $\frac{3}{8}$ in. thick web plate throughout : shear at splice 50 tons ; finished rivet diameter $\frac{3}{4}$ in.

The working stresses for plate girders, see p. xiv, are :—

Tension (F_t), tons per net sq. in.	= 9.5
Shear, web (F_w), tons per gross sq. in.	= 6.5
	= $0.684F_t$
Rivets.—Single shear (F_s)	= 6π /sq. in.
Double shear (F_{ds})	= 12π /sq. in.
Bearing (F_b)	= 12π /sq. in.
	= $1.264F_t$

Rivet Values.

Double shear = $0.44 \text{ sq. in.} \times 12 \text{ tons/sq. in.} = 5.30 \text{ tons/rivet}$

$\frac{3}{8}$ " bearing = $\frac{3}{8}" \times \frac{3}{4}" \times 12^T/\text{sq. in.} = 3.38 \text{ ,,}$

Number of rivets required = $50^T \div 3.38^T = 15,$

on each side of joint.

Number of rivets adopted to obtain an even pitch = 20

Cover Area.

Gross area of web pl. = $48" \times \frac{3}{8}" = 18 \text{ sq. in.}$

Gross area of covers = $18 \text{ sq. in.} + 5\%$
required (Symmetrical joint) = 18.9 ,,

Gross area given = $2 @ 39\frac{3}{4}" \times \frac{5}{16}" = 24.8 \text{ ,,}$

($\frac{1}{4}"$ is too thin for outside work.)

To ensure that the cover plates may clear any sectional growth of the main angles, $\frac{1}{8}$ in. is allowed, top and bottom, as a clearance.

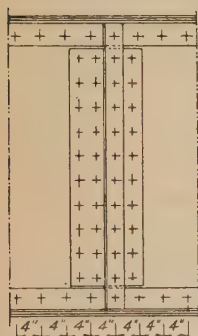
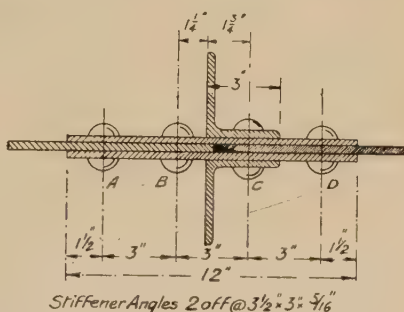


FIG. 102



Stiffener Angles 2 off @ $3\frac{1}{2} \times 3 \times \frac{5}{16}$

FIG. 103

Alternative Method. If the shear were unknown the safest method would be to design the splice to develop the full strength of the web plate.

The permissible shear stress on web = $0.684F_t$

Bearing value of 1 rivet = $\frac{3}{8}" \times \frac{3}{4}" \times 1.264F_t = 0.356F_t$

Double shear of 1 rivet = $0.44 \times 1.264F_t = 0.556F_t$

Shear value of web pl. = $48" \times \frac{3}{8}" \times 0.684F_t = 12.312F_t$

Number of rivets required = $12.312F_t \div 0.356F_t = 34.6$

This would give three rows of 13 rivets per row at 3 in. vertical pitch, and 2 in. from the centres of the outermost rivets to the toes of the flange angles.

Figs. 102 and 103. It is good practice to have a double angle stiffener placed over a web splice wherever possible. It supplies

rigidity to the structure where it is most needed, and is given as an additional safeguard.

Fig. 103 is an enlarged cross-section indicating the position of the heels of the stiffener angles relative to the joint. The rivets through the web covers were pitched at 3 in. horizontally. Rivet *C* has in addition to pass through the stiffener angle, whose rivet line for the 3-in. leg is $1\frac{3}{4}$ in. from the heel. The heels of the stiffener angles therefore take the positions marked on the drawing. There is no necessity for the angle heels and the joint to be collinear. The distance of $1\frac{1}{4}$ in. from rivet *B* to the heel of the stiffener gives sufficient clearance for the rivet die to close rivet *B*.

Fig. 104 is a magnified view of the upper portion of Fig. 102. The stiffener angles are joggled over the vertical legs of the main

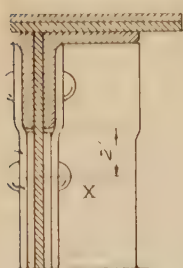


FIG. 104

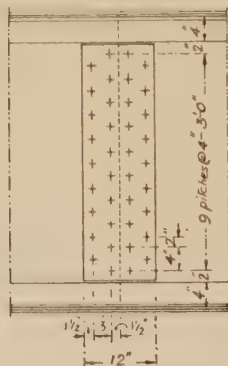


FIG. 105

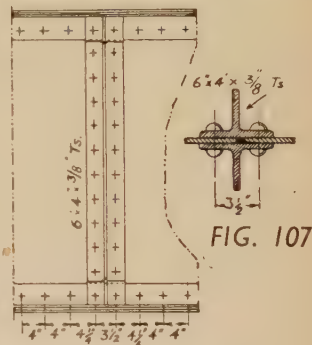


FIG. 106

angles and are ground down to fit tightly against the horizontal legs of the main angles. Rivet *X* must be kept far enough away from the higher metal of the riveted leg to allow the standard die to close rivet *X*. This distance of 2 in. was the governing dimension in laying out the vertical pitching of the cover rivets of Figs. 99 and 102. The former figure, for clearness, is shown without stiffeners.

Fig. 105 is an alternative detail to that of Fig. 99. The rivets are reeled vertically in order that at any horizontal section through the rivets there are only two holes out of the cover plate instead of the four of Fig. 99. The resulting joint is assumed by some designers to be stronger on this account, but this is questionable owing to the inclined tensile and compressive forces existing in the web plate. Reeled holes are on this account more apt to facilitate than to prevent collapse through diagonal fracture from hole to hole.

It is a trifle cheaper to have the chain riveting of Fig. 99 than the reeled riveting of Fig. 105, but there is little to choose between them either on the score of strength or expense.

Figs. 106 and 107 illustrate a web splice which is still used, but fortunately by few designers. Little can be said in its favour.

The primary idea is economy in so far that the tee acts simultaneously as a web cover for the joint and as a stiffener.

The use of this section is at variance with the recommendations of a previous paragraph, wherein it was stated that a web splice should have at least two rows of rivets on each side of the joint for lateral stiffness.

Further, the tee itself is a bad section to use as a stiffener on a plate girder. The standard spacing of the parallel lines of rivets in the table of the tee is generally different from the pitch of the main angle rivets. The main angle rivet pitch is therefore broken in its continuity by the tee. In that particular portion of the girder which is drawn the standard pitch is 4 in., but the introduction of the tee section necessitates a larger pitch of $4\frac{1}{4}$ in., then $3\frac{1}{2}$ in., and again $4\frac{1}{4}$ in. in order that their sum of 12 in. be the same as three standard pitches of 4 in.

Compare this with Fig. 102, where the angle stiffener does not interfere with the continuity of the pitch of the main flange angle rivets.

EXAMPLE OF A WEB SPLICE FOR B.M. AND SHEAR. Figs. 108 and 110

Data. Design the web splice for the 40-ft. span gantry girder of Chapter II., Vol. II. Web plate $48" \times \frac{3}{8}"$, with one-eighth of its area included in each of the flange areas. Tension flange net area is 17.2 sq. in., made up thus:— $\frac{1}{8}$ web = 2.25, angles = 9.62 and flange plate = 5.3 (item 16 of the girder calculations). Total maximum bending moment at the section is 479.7 ft. tons, with a simultaneously occurring shear of 48.7 tons (see items 42 and 43 of the girder calculations). Finished rivet diameter $\frac{15}{16}$ in. The working stresses are those stated for the previous numerical example of Fig. 99.

The term "one-eighth of the web" represents the approximation made as to the net area of the web which is effective as tension flange area (see Chapter I., Vol. II.). For this reason, and the following one also, the joint will be designed from the aspect of tension, and the compression side of the cover will be made similar to the tensile side.

In the splice the upper edges of the web plate tend to meet, and this butting action relieves the rivets and covers of stress, the covers

simply functioning as guides. The lower edges of the joint tend to leave each other, thus placing stress on the cover rivets, while the bottom covers themselves will be in tension.

$$\frac{B.M. \text{ carried by web}}{B.M. \text{ total}} = \frac{\frac{1}{8} \text{ web area}}{\text{total tensile area}} = \frac{2.25}{17.2} = 0.131.$$

$$\therefore \begin{array}{l} B.M. \text{ carried by the web} = 0.131 \text{ of total} \\ B.M. \text{ of } 479.7 \end{array} \quad \left\{ \begin{array}{l} = \text{ft. tons } 62.7. \\ = \text{in. tons } 753. \end{array} \right.$$

Since the edge angles of the top flange are assumed to form an independent horizontal girder, the centre of gravity, and therefore the neutral axis of the vertical plate girder, is midway between the flanges, as is the case in most plate girders.

The permissible tensile stress at *B*, the centre of gravity of the flange, is 9.5 tons per square inch. A fibre at *D* will have a much lower stress acting on it since it is nearer the neutral axis, i.e.,

nearer the plane of no stress. Then, by similar triangles, $\frac{\text{stress at } D}{\text{stress at } B}$

$$= \frac{Dd}{Bb} = \frac{OD}{OB} = \frac{13\frac{1}{2}}{23}. \quad \text{Whence the stress at } D = \text{stress at } B \times$$

$(13\frac{1}{2} \div 23) = 9.5 (13\frac{1}{2} \div 23) = 5.67 \text{ sq. in.}$, which is the average stress acting at *D*.

Similarly, if the *D.S.* and rivet-bearing values on $\frac{3}{8}$ in. web at *B*

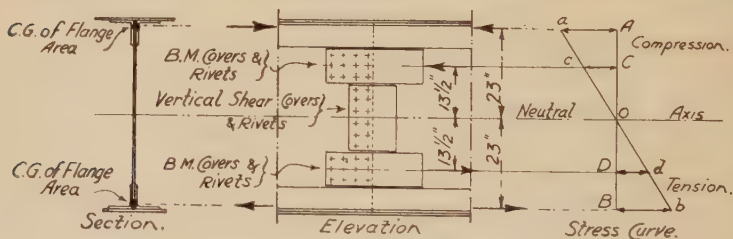


FIG. 108

are respectively 8.28 tons and 4.22 tons (see rivet values on p. xv),

those at *D* will have the values of $8.28 \times \frac{13\frac{1}{2}}{23}$ and $4.22 \times \frac{13\frac{1}{2}}{23}$,

namely, 4.86 and 2.48 tons acting on each rivet, the latter values being the mean rivet values of all the rivets in cover *D*. The row nearer the neutral axis will have a lesser value, and the outer row a greater value than the mean.

The magnitude of the tensile force in covers *D* is equal to that of the compressive force in covers *C*. These two forces form a couple, whose lever arm is twice $13\frac{1}{2}$ in., counteracting the *B.M.* in the web.

∴ Force $C = \text{Force } D$	$= 753 \text{ in. tons} \div 27''$	$= \text{tons}$	28
Number of rivets required			
in each set of covers	$= 28 \div 2.48$	$= 11.3, \text{ i.e., } 12$	
Area required for tensile			
covers	$= 28 \div 5.6$	$= \text{net sq. in.}$	5
Given 2 pls. @ $9'' \times \frac{1}{2}''$,			
area net	$= 2 (9'' - 3 \times \frac{15}{16}'') \times \frac{1}{2}'' =$	$,,$	6.2

The compression or upper covers are made similar to the tensile ones.

Mid Vertical Covers. The rivets in these are calculated on the assumption that they carry only vertical shear, and since shear is practically constant or uniform throughout the girder depth, the

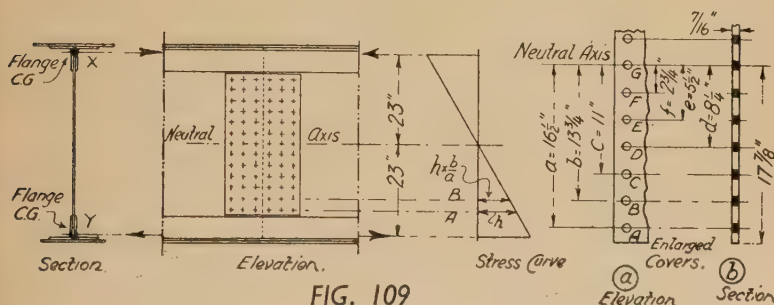


FIG. 109

full rivet value of 4.22 tons is used. As will be seen from the following alternative design, these centre rivets must also partake of the bending moment, and therefore this method of calculation is not theoretically correct. Further, the combined web splice has two horizontal joints in the cover plates, where the horizontal shear is not carried across into the adjoining cover.

Vertical shear at section	$= \text{tons}$	48.7
Given 12 rivets on each side of joint @ 4.22 tons		
(bearing $\frac{3}{8}''$)	$= ,,$	50.6
Gross area of covers required $= \text{vertical shear} \div$		
permissible stress	$= 48.7 \div 6.5$	$= \text{sq. in.}$ 7.5
Gross area of covers given	$= 2 @ 17\frac{3}{4}'' \times \frac{1}{2}''$	$= ,,$ 17.5

The thicknesses of the inner vertical covers are kept the same as those at C and D to avoid double joggling of the angle stiffeners at the joint.

ALTERNATIVE METHOD FOR A B.M. AND SHEAR WEB SPLICE
Figs. 109 and 111

Data. As for previous example.

In this case the cover on either side of the web is in one piece and not in three. The rivets are, therefore, subjected to a horizontal pull due to bending plus a vertical load due to shear. Where the web carries *B.M.* as well as shear, it is always the former which controls the design. In fact, no great harm would occur in the majority of cases were shear to be neglected altogether.

At *Y*, the centre of gravity of the bottom flange, the permissible tensile stress is 9.5 tons per net square inch. The working loads per

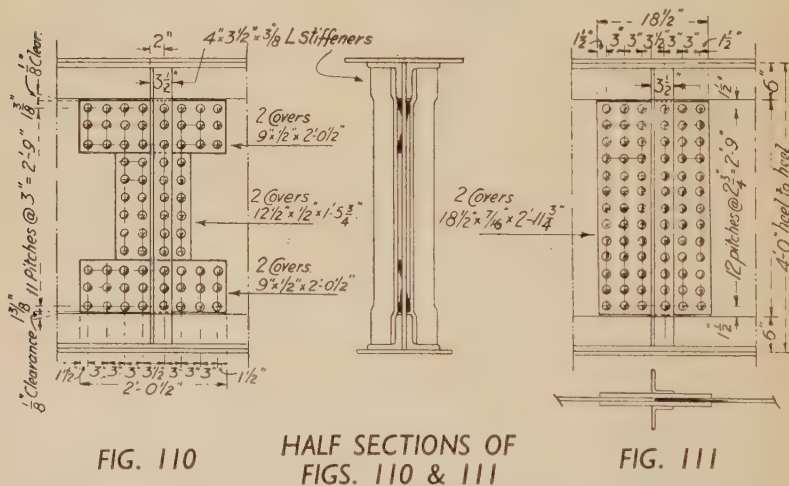


FIG. 110

HALF SECTIONS OF
FIGS. 110 & 111

FIG. 111

rivet at *Y* are:—*D.S.* = 8.28 tons and $\frac{3}{8}$ in. bearing on web = 4.22 tons. From the stress curve the tensile stress at $17\frac{7}{8}$ in. from the neutral axis (*i.e.*, the extreme fibre of the cover) is $9.5 \times 17\frac{7}{8} \div 23 = 7.38$ tons/sq. in. Similarly, the working load on rivet *A* must not exceed $4.22 \text{ tons} \times 16\frac{1}{2} \div 23 = 3.03$ tons.

The web tends to open out at the bottom due to the bending action, but is restrained by the covers. The covers pick up their load from the rivets, and therefore rivet *A* offers a resisting moment about the neutral axis, the value of which is the actual load on rivet *A* multiplied by lever arm *a*. Similarly, rivet *B* offers a resisting moment of rivet load at *B* multiplied by its lever arm *b*, and so on.

Let the horizontal load on rivet *A*, due to bending, be *h* tons, then that on rivet *B* is $h \times (b \div a)$ tons, and that on rivet *C* is $h \times$

($c \div a$), etc.; i.e., proportional to the rivet distance from the N.A. as shown in the stress curve above; or, in tabular form:—

Rivet.	Actual Horizontal Load.	Lever Arm.	Moment.
<i>A</i>	h	a	$ha = ha^2 \div a$
<i>B</i>	$h \times (b \div a)$	b	$hb^2 \div a$
<i>C</i>	$h \times (c \div a)$	c	$hc^2 \div a$
<i>D</i>	$h \times (d \div a)$	d	$hd^2 \div a$
	etc.		

The total resisting moment of one half row

$$= \frac{h}{a} \Sigma a^2 + b^2 + c^2 + d^2 + \dots + g^2.$$

For one complete row the moment is $\frac{h}{a} \times 2\Sigma a^2 + b^2 + \dots + g^2$.

The numerical value of this (remembering that h should not exceed

$$3.03 \text{ tons}) \text{ is } \frac{3.03}{16\frac{1}{2}} \times 2 (16\frac{1}{2}^2 + 13\frac{3}{4}^2 + \dots + 2\frac{3}{4}^2 + 0^2) = \frac{3.03}{16\frac{1}{2}} (1,376.36) = 253 \text{ in. tons : see Figs. (a) and (b).}$$

To increase the resisting moment of the rivets these were pitched vertically at the minimum spacing of $3d = 2\frac{3}{4}$ ". See further remarks on this point in items **44** and **46** of the main girder calculations.

Bending moment carried by web = in. tons 753

∴ Number of rows of rivets required on each side of joint = $753 \div 253 = 3$

Vertical load per rivet due to shear = v
= 48.7 tons $\div$ 39 rivets = tons 1.2

Since the B.M. of 753 = 3 rows at $\frac{h}{a} \times$

$$2\Sigma a^2 + b^2 + \dots + g^2 = \frac{h \times 3}{16\frac{1}{2}} (1,376.36)$$

∴ The horizontal load on rivet $A = h = \frac{753 \times 16\frac{1}{2}}{3 (1,376.36)} = \text{tons } 3$

By the parallelogram of forces, resultant load on rivet $A = \sqrt{h^2 + v^2} = \sqrt{10.44} = \text{tons } 3.23$

Permissible load on rivet $A = 3.03$ tons. The overload on rivet A is thus 6 per cent. but may be neglected in view of the excess

large material which exists at this point. See item 47 of main girder calculations.

Cover Plate. Gross M of I of one plate, Fig. 109 (b)

$$= \frac{1}{12} \times \frac{7}{16} \times (35\frac{3}{4})^3 = \text{in.}^4 \quad 1666$$

Inertia of holes = $\Sigma (\text{area}) \times (\text{distance})^2$

$$= (\frac{11}{16} \times \frac{7}{16}) [2(16\frac{1}{2}^2 + 13\frac{3}{4}^2 + \dots + 2\frac{1}{2}^2 + 0^2)]$$

$$= \frac{11}{16} \times \frac{7}{16} \times 1,376.36 = \text{,,} \quad 564$$

$$\text{Net moment of inertia } I \text{ of one cover} = \text{,,} \quad \underline{1102}$$

$$\text{Net moment of inertia } I \text{ of two covers, back and front} = \text{,,} \quad 2204$$

$$\text{Net section modulus } Z \text{ of two covers} = I/y$$

$$= I \div 17.875 = 2204 \div 17.875 = \text{in.}^3 \quad 123$$

$$\text{Extreme fibre stress, tension} = B.M. \div Z =$$

$$753 \div 123 = \text{tons/sq. in.} \quad 6.1$$

$$\text{Permissible stress in tension at } 17.875'' \text{ from the neutral axis was} \quad \text{,,} \quad 7.38$$

Vertical shear stress in covers

$$= 48.7 \text{ tons} \div \text{gross area of two covers,}$$

$$= 48.7 \div 2 \times 35\frac{3}{4} \times \frac{7}{16} = \text{,,} \quad 1.55$$

Permissible shear stress in covers as for main

$$\text{web} = \text{,,} \quad 6.5$$

Because this crane girder and web splice have to withstand lateral forces in addition to vertical forces (see Chap. XI) $\frac{7}{16}$ -in. thick covers are used. If only vertical loading had to be considered then $\frac{3}{8}$ -in. thick covers are satisfactory with an extreme fibre stress of 7.12 tons per sq. in. against the permissible of 7.38 tons per sq. in.

Rivet Modulus. The formula for the moment of resistance of the rivets, newly found, is the $\frac{M}{I} = \frac{f}{y}$ form slightly disguised.

Thus the $B.M.$ of 753

$$= 3 \text{ rows } @ \frac{h}{a} \times 2\Sigma a^2 + b^2 + c^2 + \dots g^2$$

$$= \frac{h}{a} \Sigma 6a^2 + 6b^2 + \dots 6g^2$$

$$= \frac{h\Sigma \text{distances}^2}{\text{extreme distance}}$$

Now h , the total horizontal load on the rivet, = rivet area $A \times f$, the stress per square inch, where A = the shear area (single or

double) or the bearing area (*i.e.*, diameter $\times$ plate thickness), and f is respectively the shear stress in tons per square inch (single or double) or the bearing stress. Hence

$$\frac{h\Sigma \text{ dist.}^2}{\text{extreme dist.}} = \frac{f \times A\Sigma \text{ dist.}^2}{\text{extreme dist.}} = \frac{f \times I}{y}, \text{ since } I = A\Sigma \text{ dist.}^2$$

$$\text{But } Z = \frac{I}{y} = \frac{A\Sigma \text{ dist.}^2}{\text{extreme dist.}} = \frac{A(3 \times 1,376.36)}{16.5} = 250.2A.$$

$$\therefore \text{The stress in the extreme rivet} = f = M \div Z = 753 \div 250.2A \\ = \frac{3}{A} \text{ tons/sq. in.}$$

The horizontal load on the extreme rivet = stress in tons/sq. in. $\times$ area in sq. in. $= \frac{3}{A} \times A =$ as before, 3 tons.

Since the shear is double,
then $A = 2 \times 0.7854 \times (\frac{15}{16})^2 = \text{sq. in.} \quad 1.381$

Whence the horizontal
shear stress $= 3 \div 1.381 \text{ sq. in.} = \text{tons/sq. in.} \quad 2.17$

Similarly, the bearing area
on the $\frac{3}{8}$ " web $= \frac{3}{8} \times \frac{15}{16} = \text{sq. in.} \quad 0.352$

While the horizontal bear-
ing stress $= 3 \div A = 3 \div 0.352 = \text{tons/sq. in.} \quad 8.5$

The vertical shear stress $= 1.2 \div 1.381 \text{ sq. in.} = \text{,,} \quad 0.87$

The vertical bearing stress $= 1.2 \div 0.352 \text{ ,,} = \text{,,} \quad 3.41$

The resulting bearing
stress $= \sqrt{(8.5^2 + 3.41^2)} = \text{,,} \quad 9.15$

The permissible bearing stress at the *C.* of *G.*
of the flanges $= \text{,,} \quad 12$

and the permissible bearing stress at extreme
rivet $= 12 \times \text{ratio of the distances from}$
N.A. $= 12 \times 16.5 \div 23 = \text{,,} \quad 8.61$

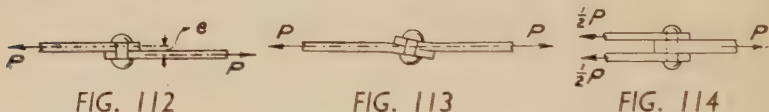
The overload $= 9.15 - 8.61$ in tons per
square inch, or, as before 6%

CHAPTER VIII

ECCENTRIC RIVETED CONNECTIONS

WITH the exception of the web splice carrying bending moment, all the previous examples of riveted joints carried, or were assumed to carry, direct load only. Cases arise where the rivets have to counteract bending moment in addition to direct load, as happened in the last examples of the web splice. If the bending moment is small it is neglected and only the direct load is considered. The simple lap joint is a case in point.

The resultant pull runs along the centre of gravity of each plate (Fig. 112), and at the joint these lines of force are at a distance e apart. The couple of $P.e$ acts upon the joint and tends to bend the



plates so that the two centre of gravity lines become collinear, as in Fig. 113.

The rivet shanks are now also in tension, as there is a lever action trying to force the rivet heads apart. Also, the thicker the plates the greater is the eccentricity e , and therefore the greater the moment of the couple. However, the rivet diameter increases with the plate thickness, so that, relatively, the bending stresses due to eccentricity are approximately the same on small-diameter rivets as on large-diameter rivets.

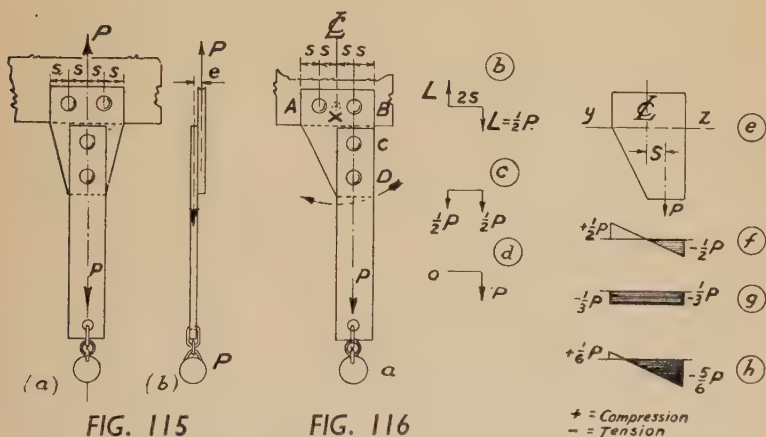
If this bending moment $P.e$, due to the couple, is to be neglected, as it is in practice, then there is no necessity for a great refinement of calculation. Further, when it is known that such a bending moment occurs it is really false economy to give the exact and minimum number of rivets required to carry the direct pull P in single shear or in bearing.

The double-butt strap joint has no such bending action provided the plates take their proper share of the load. This in itself is open to question if one desires to cavil over small things. The rivets may not fill the holes completely, or the plates may not all have the same elastic properties, and thus it is quite probable that

a greater proportion than $\frac{1}{2}P$ may be flung on to one or other of the two outer plates of Fig. 114.

Fig. 115 is as near the ideal connection as can be attained with lapped plates. The only eccentricity is the unavoidable and negligible one due to plate thicknesses, Fig. *b*. The load on each of the four rivets is $\frac{1}{2}P$.

In Fig. 116 the line of pull is no longer coincident with the axis of symmetry of the upper pair of rivets, whose centre of gravity lies at *X*. Assume the uppermost member rigid and a pin driven in at *X* replacing the two upper rivets, then the vertical flat and gusset would swing, pendulum fashion, as indicated by the feathered arrow. Rivet *A*, therefore, is pushed upwards, while rivet *B* is pulled



downwards, both with a force of, say, L tons. In other words, there is a clockwise couple acting on the rivets equal to $L \times$ distance $AB = 2SL$, Fig. *b*. The cause of this couple is the bending moment due to the eccentricity of P with respect to point *X*, the centre of gravity of rivets *A* and *B*. Therefore, $2SL =$ bending moment $= P \times S$, hence $L = \frac{1}{2}P$. In addition to this bending moment the upper rivets carry the direct load P equally between them, as indicated in Fig. *c*.

The resultant load on each of the upper pair of rivets is the summation of the loads of Figs. *b* and *c*, and is given in Fig. *d*. The latter figure shows that rivet *B* carries all the load of P tons, whereas the load on rivet *A* is nothing. Rivet *B* may be seriously overstressed, carrying, as it does, the load of either rivet *C* or *D*.

Similarly the centre line of the gusset plate does not coincide

with the line of action of the external load P , so that the plate has to withstand a direct load P + a *B.M.* of $P.S.$, Fig. *e*. The resulting stresses at any section yz , where the centre line is CL , are found as follows. Let the rivet pitch $= 2S = 3"$ and let the gusset plate thickness be $\frac{1}{8}$ in. Tension is given the negative sign.

Cross-sectional area of

$$\text{gusset pl. at line } yz = 6" \times \frac{1}{8}" = 3 \text{ sq. in.}$$

Direct tensile load per sq.

$$\text{in. on line } yz = P \div 3 = (\text{Fig. } g), -\frac{1}{3}P.$$

The *B.M.* on the gusset is

$$P \times \text{eccentricity } S = 1\frac{1}{2} P \text{ in. tons.}$$

Section modulus along line

$$yz, (\frac{1}{8}bd^2) = \frac{1}{8} \times \frac{1}{2} \times 6^2 = 3 \text{ in.}^3$$

Extreme fibre stress =

$$B.M. \div \text{section modulus} = 1\frac{1}{2}P \div 3, (\text{Fig. } f) = \pm \frac{1}{2}P \text{ tons/sq. in.}$$

(Right edge in tension, left edge in compression.)

Adding the direct and bending stresses together (Fig. *h*), it is found that the left edge is stressed to $-\frac{1}{3}P + \frac{1}{2}P = +\frac{1}{6}P$ tons per square inch, and the right edge is stressed up to $-\frac{1}{3}P - \frac{1}{2}P$, or $-\frac{5}{6}P$ tons per square inch, where $+$ is the compression sign. As a check upon the calculations the average of these two fibre stresses should be equal to the direct load intensity, viz. $-\frac{1}{3}P = (-\frac{5}{6}P + \frac{1}{6}P) \div 2$.

Fig. 117. The rivets in the flat bar are now eccentric. The upper rivets in the gusset plate each carry a load, in tons, of $\frac{1}{2}P$, direct load only.

The flat bar rivets are not coincident with the line of pull of P , and are thus subjected to both a direct and a bending stress. The direct load for either rivet C or D is $\frac{1}{2}P$ tons. The bending moment on these rivets is $P \times e$, where e is the eccentricity of the line of pull from the gravity centre X of the rivets. The counteracting couple of the rivets is $L \times 2S$, where L is the force per rivet and $2S$ is the pitch. In order to find the sense of L , assume both gusset and upper member rigid, and that a pin at X replaces rivets C and D . The flat will swing counter-clockwise about X in order that the weight may be directly under the pin.

As before, $P.e = 2LS$, or $L = P.e \div 2S$. The total load on either rivet C or D is found by adding, as vector quantities, $\frac{1}{2}P$ to L . As a numerical example let $P = 5$ tons, $e = \frac{3}{8}$ in. and rivet pitch $2S = 3$ in.

$$B.M. = P.e = 5 \times \frac{3}{8} = \text{in. tons} \quad 1\frac{7}{8}$$

$$\text{Couple} = 2LS = 2L \times 1\frac{1}{2} = \text{,,} \quad 3L$$

$$\text{Since } 3L = 1\frac{7}{8} \therefore L \text{ is } \frac{5}{8} \text{ ton.}$$

$$\text{The direct load per rivet is } 5 \div 2 = \text{tons} \quad 2\frac{1}{2}$$

The resultant load on either C or $D = \sqrt{2.5^2 + \frac{5^2}{8}} = 2.58$ tons, Fig. 118.

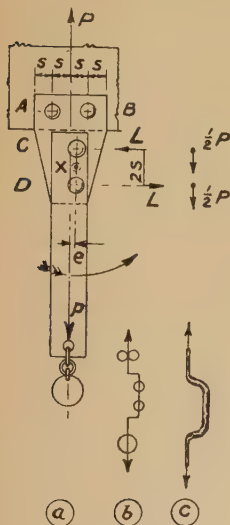


FIG. 117

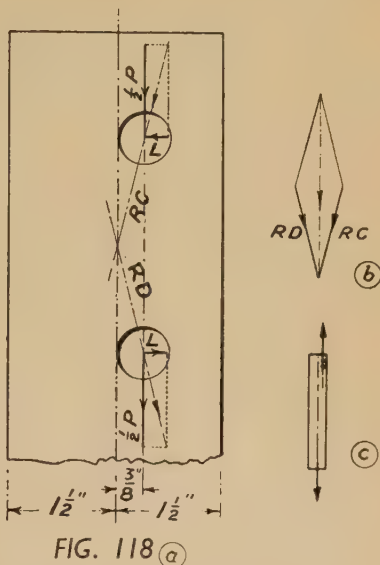


FIG. 118 (a)

Fig. 118. There is no bending on the vertical bar of Fig. 117, because the line of pull is coincident with the centre of gravity line. This statement seems to confuse beginners as will be appreciated from Fig. 118c, which shows the vertical bar together with the student's usual conception of the forces acting on it. The forces, however, are incompletely shown because force L has been left out at the rivets. The proof that there is no bending on the vertical bar is obtained at once by finding the resultant of forces RD and RC , Fig. *b*. This resultant is found to be 5 tons exactly, acting on the flat's centre line, Fig. *a*, i.e., the resultant is exactly equal, but opposite in sense to P .

Similarly there is no bending on the gusset, although at first sight this case appears to be similar to that of Fig. 116e. The downward force on the gusset is certainly applied off centre through the rivet holes by the rivets C and D , but the horizontal forces L bring the actual load line back on to the axis of symmetry of the gusset.

The analogy to the cranked tie-bar is illustrated by diagrams *b* and *c* of Fig. 117.

Fig. 119. Here the pull is central with the gusset rivets and coincident with the flat bar rivets. All four rivets will carry,

therefore, a direct load only of $\frac{1}{2}P$ tons per rivet. The flat bar is eccentric from the line of pull by an amount $e = \frac{3}{8}$ " , and it is the bar, in this case, which is subjected to direct and bending stresses. Due to bending, caused by this eccentricity, the left-hand edge of the flat tends to compress while the right-hand edge tends to elongate. This follows at once from the pin test; *i.e.*, by imagining pin *X* placed on the centre of gravity line of the structural element

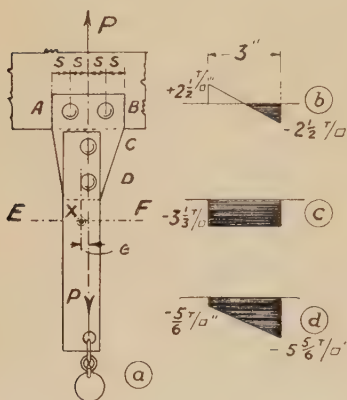


FIG. 119

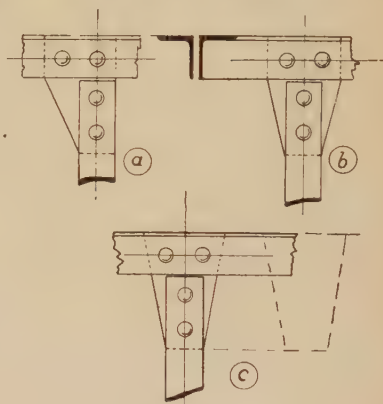


FIG. 120

under consideration and then sketching the path of rotation of the element about this centre.

Assume the bar to be a $3'' \times \frac{1}{2}''$ flat, to find the stress distribution across any section such as *EF*.

Cross-sectional area at *EF* = $3'' \times \frac{1}{2}''$ = sq. in. $1\frac{1}{2}$

Stress due to direct load, Fig. *c*, = $P \div \text{area}$
= $-5 \text{ tons} \div 1\frac{1}{2}$ = tons/sq. in. $-3\frac{1}{3}$

Bending moment on the bar = $P \times e$ =
 $5\pi \times \frac{3}{8}''$ = in. tons $1\frac{7}{8}$

Section modulus *Z* of bar = $(\frac{1}{6}bd^2) =$
 $\frac{1}{6} \times 3 \times \frac{1}{2} \times 3^2$ = in.³ $\frac{3}{4}$

Extreme fibre stress due to *B.M.*, Fig. *b* =
 $B.M. \div Z = 1\frac{7}{8} \div \frac{3}{4}$ = tons/sq. in. $\pm 2\frac{1}{2}$

Total extreme fibre stress, Fig. *d* =
 $-3\frac{1}{3} \pm 2\frac{1}{2}$ = , $-5\frac{5}{6}$ & $-5\frac{1}{6}$.

As a verification the average of the two last stresses should give the direct stress, *i.e.*, $6\frac{2}{3}\pi \div 2 = 3\frac{1}{3}\pi$.

The stress in the bar, when central with the load line, is $3\frac{1}{3}$ tons per square inch, but by giving the eccentricity of $\frac{3}{8}$ in. this stress

has almost been doubled in the case of the extreme right-hand fibre.

Fig. 120 illustrates common roof truss details. Case *a* is bad, since the top right-hand gusset rivet is seriously overloaded, as proved in regard to Fig. 116. No defence can be offered for its continual use, not even on the plea of economy. It requires five cuts to make this gusset.

Detail *b* is much superior, as it has all the rivets centrally loaded. The gusset, however, requires six cuts.

Fig. *c* is the ideal connection for this case because the rivets are centrally loaded while the gusset plate has only four cuts. The gusset can be cut from one strip (see dotted lines) with practically no waste, only two end-pieces being left over. Compare this with the scrap entailed in making gusset plate *a*, where every gusset has a small triangular waste piece.

Fig. *b* appeals to some designers more than Fig. *c*, but once the gusset is riveted in between the main rafter angles they both look the same in the finished roof truss.

CRANE BRACKET. Figs. 121 to 124

The worst case of loading occurs when the crane and its load are drawn up close to the column. This gives the maximum vertical load. The lateral thrust due to the cross travel of the crab can be neglected, because the crab, if carrying full load, must go very slowly when in such close proximity to the end of its travel.

The question now arises as to how much load comes on to each bracket plate. If each bracket plate was independent of its neighbour, then either plate would, in turn, have to support the entire reaction due to the crane and its load. By using a diaphragm one side plate cannot deflect without taking its neighbour with it, and its neighbour cannot deflect unless it also participates in the carrying of the load, since deflection is proportional to load. There is another factor which is sometimes considered, and that is the frequency of this particular case of loading. The crane wheels will be often in the position shown, but it will be seldom that they will be called upon to carry the maximum load with the crab drawn in tightly against the bracket. The fact that maximum load is practically synonymous with maximum bulk explains this. The gantry girder was designed for the worst possible cases of loading without consideration as to the laws of chance, and to be consistent the vertical brackets should be designed to meet the maximum load. Only the question of the diaphragm is now left. Conservative designers make each bracket plate strong enough to carry all the reaction.

Others, in competitive work, often assume that the unloaded plate relieves the loaded plate of a certain amount of load, the maximum load on the loaded plate being variously estimated at one-half, three-quarters and seven-eighths of the maximum girder reaction. The half is decidedly risky in view of the shallow depth of diaphragm and the resulting small landing left for pitching rivets therein, because the rivets through the plate into the diaphragm must, at the very least, be sufficient in number to carry that portion of the load which is assumed transferable to the idle plate.

In the following example the course chosen is neither too conservative nor too venturesome, as the active plate will be assumed to carry three-quarters of the live load reaction, care being exercised that the diaphragm is of the requisite depth for rivet spacing.

Numerical Example. Design a crane bracket for the 20 ft. span joist and channel gantry girder of Chapter XI., but attached to a slightly different type of column.

Dead load reaction from girder, at each bracket plate (item 1, Chapter XI) = $\frac{1}{2}$ of 0.8^{r}	= tons	0.4
Live load reaction plus impact on one complete bracket (item 2, Chapter XI) = 18 tons.		
Assume live load on bracket plate as being $\frac{3}{4}$ of 18^{r}	= „	13.5
Total load P , Fig. 121, on one plate = $0.4^{\text{r}} + 13.5^{\text{r}}$	= „ say	14.0
Load carried to "idle" plate by diaphragm is $\frac{1}{4}$ of 18^{r}	= „	4.5
Permissible tensile stress per square inch, see girder calculations	= „	10
Max. $B.M.$ on rivet group about the centre of gravity $X = P \times e = 14^{\text{r}} \times 13\frac{1}{2}''$	= in. tons	189

The group of rivets, therefore, carries a direct vertical load of 14^{r} plus a bending moment of 189 in. tons. The direct vertical load is shared equally by all the rivets and hence vertical load per rivet is

$$14^{\text{r}} \div 14 \text{ rivets} = 1^{\text{r}}.$$

If the plate rotated under the bending moment it would push that portion of rivet A which projects out of the column upwards and towards the right, the thrust being at right angles to the radial line XA , whose length is a . This thrust L would have the same numerical value at Q , but would be downwards and towards the left, again acting at right angles to the radial line XQ of length $q = a$. Now the nearer the neutral axis or centre of gravity the less is the thrust on the rivets, hence the load on rivet B will be $L(b/a)$ and on rivet C it is $L(c/a)$, as explained in the previous chapter.

A table similar to that employed in the calculations for the web splice carrying shear and bending moment of the preceding chapter can now be drawn up.

Rivet.	Load.	Radial Lever Arm.	Resisting Polar Moment = Load $\times$ Lever Arm.	
<i>A</i>	<i>L</i>	<i>a</i>	<i>La</i> or	$La^2 \div a$
<i>B</i>	$L \times \frac{b}{a}$	<i>b</i>		$Lb^2 \div a$
<i>C</i>	$L \times \frac{c}{a}$	<i>c</i>		$Lc^2 \div a$

etc.

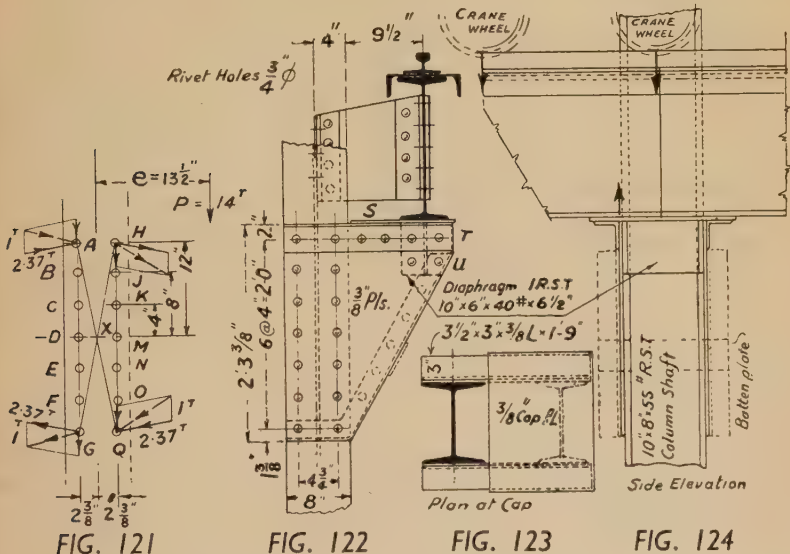


FIG. 121

FIG. 122

FIG. 123

FIG. 124

$\therefore$ The total resisting moment = $\frac{L}{a} \Sigma a^2 + b^2 + c^2 + \dots n^2 + o^2 + q^2$, which must equal the external B.M. of $P.e = 189$ in. tons.
 $\therefore$ Thrust L , due to eccentricity = $189 \times a \div \Sigma a^2 + b^2 + c^2 + \dots n^2 + o^2 + q^2$.

By Euclid I — 47 : square on hypotenuse = sum of squares on other two sides. $\therefore a^2 = 12^2 + 2\frac{3}{8}^2$, (Fig. 121), $b^2 = 8^2 + 2\frac{3}{8}^2$ and $c^2 = 4^2 + 2\frac{3}{8}^2$.

Whence $L = 189 \times 12.23 \div [4(12^2 + 8^2 + 4^2) + 14(2\frac{3}{8})^2]$
 $= 189 \times 12.23 \div 975 = \text{tons } 2.37$

The total load on any rivet is found by drawing the parallelogram of forces for that rivet. Note that the arrow-heads of the forces must either be both away from the rivet or both towards the rivet. Total load on the rivet *A* or *G* is the resultant of 1^T and $2.37^T = 2.38^T$. Total load on the rivet *H* or *Q* is the resultant of 1^T and $2.37^T = 2.74^T$.

Observe that the angle between the 1^T and 2.37^T loads is acute for rivets *H* and *Q* and obtuse for rivets *A* and *G*.

From page xv the value per $\frac{3}{4}$ in. diameter rivet is 2.65^T , *S.S.* and 3.38^T for $\frac{3}{8}$ in. bearing.

There are only two rivets, *H* and *Q*, which are slightly overstressed ($2.74^T - 2.65^T = 0.09^T$ or 3 per cent.) ; all the remaining 12 rivets are under the permissible load of 2.65 tons per rivet. In view of the fact that the exact distribution of the load to the bracket plate is a matter of conjecture the riveting may be accepted as being quite satisfactory.

With the wheels as shown in Fig. 124, part of the load finds its way into the left side-bracket plate through the diaphragm, and the remainder through the horizontal $3\frac{1}{2}'' \times 3'' \times \frac{3}{8}''$ angle under the loaded side of the cap plate of Fig. 123. The two rivets at *T* are thus in double shear and $\frac{3}{8}$ -in. bearing, while the two at *S* and the two at *U* are in single shear and $\frac{3}{8}$ -in. bearing. The extreme left-hand rivet at *S*, however, can transfer but little load into the side plate.

The value of the riveting is therefore 2 @ 3.38^T , *i.e.*, $\frac{3}{8}$ -in. bearing at *T* + 4 @ 2.65^T , *i.e.*, *S.S.* at *S* and *U* = 17.36^T , which is in excess of the load *P* of 14^T carried by these rivets.

Plate Calculations.

Gross moment of inertia of side plate at line

$$HQ = \frac{1}{12}bd^3 = \frac{1}{12} \times \frac{3}{8} \times 27^{\frac{3}{8}3} = \text{in.}^4 \quad 641.6$$

M. of *I.* of the holes about the N.A. = Σ area

$$\times \text{dist.}^2 = (\frac{3}{8} \times \frac{3}{4}) \times 2(12^2 + 8^2 + 4^2) = \text{,,} \quad 126.0$$

$\therefore$ Net *M.* of *I.* of plate along line *HQ* =

$$\text{difference} = \text{,,} \quad 515.6$$

$\therefore$ Net section modulus *Z* = " $I \div y$ " =

$$515.6 \div \frac{1}{2} \text{ of } 27^{\frac{3}{8}3} = \text{in.}^3 \quad 38$$

The *B.M.* at *HQ* line = $14^T(13\frac{1}{2}'' - 2\frac{3}{8}'')$ = in. tons. 155.8

Extreme fibre stress calculating with the net

$$Z = M \div Z = 155.8 \div 38 = \text{tons/sq. in.} \pm 4.1$$

The bending moment at the *HQ* line is the maximum on the plate and, following a straight line law, the moment decreases gradually to zero as the plate is traversed towards the load line of *P*.

The plate is also safe against shearing along the vertical line HQ . Shearing force $= P = \text{tons}$ 14
 $\therefore$ Shear stress $= 14^{\text{r}} \div \text{gross area of } 27\frac{3}{8}'' \times \frac{3}{8}'' \text{ thick.}$ $= \text{tons/sq. in. } 1.36$

The largest vertical shear stress intensity will be encountered in the region of the arrow which points towards S in Fig. 122, because at this point most of the load will have now entered the plate. The depth, and therefore the shear area of the plate, is approximately one-half of that at HQ , consequently the vertical shear stress is about twice that at HQ , viz., about 2.72 tons per square inch.

Sometimes an edge angle ($2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$ in this case) is riveted to the plate's compression edge to stiffen it against buckling. The maximum pitch of the connecting rivets must not exceed $12t$ or $16t$; see text to Fig. 44. A batten plate is frequently riveted across the faces of the two edge angles, Figs. 122 and 124, as a further precaution against buckling by lowering the l/k value of the angle. The edge and cap angles if well riveted may be included when calculating the section modulus of the bracket side plate.

Another Numerical Example. Figs. 125 to 127 show a type of bracket sometimes used when the web of the column shaft is at right angles to the crane girder. The upper rivets of the rows marked B and C are in axial tension and, as some designers do not like to have rivets in tension, these rivets are sometimes replaced by bolts; see text to Fig. 66. Alternatively, the detail shown on Fig. 128 may be employed.

The thickness of the bracket plate is fixed by three things:—
 (1) The rivet values at D transferring the load from the cap into the vertical plate. (2) Section modulus for bending. (3) The vertical shearing stress on the plate.

Data as for previous example.

Rivet values, $\frac{3}{4}$ -in. diameter holes. $S.S. = 2.65^{\text{r}}$, $D.S. = 5.30^{\text{r}}$ and $\frac{5}{8}$ -in. bearing $= 5.63^{\text{r}}$ per rivet.

Load on bracket $=$ dead load reaction from two girders $+ \text{live load and impact} = 2 \times 0.4^{\text{r}} + 18^{\text{r}} = 18.8^{\text{r}}$.

The weight of the bracket itself, about $1\frac{1}{4}$ cwt., is neglected.

Riveting at Row D. Cap to plate.—The four rivets immediately under the girder are in $D.S.$ and $\frac{5}{8}'' B = 4 \times 5.30^{\text{r}} = 21.20^{\text{r}}$. This is safe since it is greater than 18.8^{r} , the sum of the end loads from the loaded and unloaded girders. Only one vertical plate supports these loads now, and not two as in previous example.

Vertical Plate.

The $B.M.$ at the rivet

$$\text{line } A \quad = 18.8^{\text{r}} \times 7\frac{1}{2}'' \quad = \text{in. tons} \quad \underline{141}$$

The load on the next outer rivet is $2.56 \times 10.5 \div 13.5$. These thrusts all act horizontally, *i.e.*, at right angles to the vertical radial line.

The direct load per rivet is $18.8^T \div 10$, acting vertically downwards	= tons	1.88
Resulting load on outermost rivets	$= \sqrt{(1.88^2 + 2.56^2)} =$ „	3.17
Permissible load on outermost rivets	$= D.S. \text{ value per rivet of}$ „	5.30

The *A* row of rivets is amply safe, but the number is fixed by the number of *B* and *C* rivets required, with which rows the *A* rivets are reel-pitched.

Riveting at Rows B and C. As in the case of the *A* rivets take the distances from the *XX* line, the centre of gravity line of the group—a further discussion on this point is given later in connection with Figs. 129 to 132. (Where it is shown that the upper rivets are under axial tension, but the bottom rivets, in actual fact, are relieved of axial load since the bottom ends of the bracket angles are compressed against the column face.)

$$B.M. \text{ at column face} = 18.8^T \times 9\frac{1}{2}'' = \text{in. tons} \quad 178.6$$

Axial tensile load on each of the topmost pair of rivets

$$= \frac{B.M. \times \text{extreme radial length}}{\Sigma \text{ distance}^2}$$

$$= \frac{178.6 \times 15}{4 \times 3^2 + 6^2 + 9^2 + 12^2 + 15^2} = \text{tons} \quad 1.35$$

Direct vertical shearing load on each of the *B* and *C* rivets $= 18.8^T \div 22 \text{ rivets} =$ „ 0.85

$\therefore$ Axial tensile stress on each of the topmost pair of rivets $= f_t = 1.35^T \div 0.44 \text{ sq. in.} =$ T/sq. in. 3.07

Direct shearing stress on each of the topmost pair of rivets $= f_s = 0.85^T \div 0.44 \text{ sq. in.} =$ „ 1.93

From the theory of principal stresses (see text-books on Strength of Materials) the maximum shear stress *v* and the maximum tensile stress *f* on each of the uppermost pair of rivets are :—

$$v = \sqrt{[(\frac{1}{2}f_t)^2 + f_s^2]} = \sqrt{[(\frac{1}{2} \times 3.07)^2 + 1.93^2]} = \text{T/sq. in.} \quad 2.47$$

$$f = \frac{1}{2}f_t + \sqrt{[(\frac{1}{2}f_t)^2 + f_s^2]} = \frac{1}{2}f_t + v$$

$$= \frac{1}{2} \text{ of } 3.07 + 2.47 \quad . \quad . \quad . \quad . \quad . = \text{ „} \quad 4.01$$

As stated in the following article, the permissible axial tensile stress on rivets is $5^T/\text{sq. in.}$, or 2.21^T per $\frac{3}{4}$ in. dia. rivet.

The standard centre of holes in the joist flange of $4\frac{3}{4}$ in. has been altered to $4\frac{5}{8}$ in. to suit the holing of the angles. The reverse shel

angle, $6'' \times 4'' \times \frac{1}{2}''$, of Fig. 127, may be used in detail (Fig. 125a) as dotted, since it adds considerably to the bracket strength (the resisting moment of its rivets can be taken into account, but this is not usually done).

Fig. 127. The bracket angles of Fig. 125a were, at one time, cut, kneed and smith welded at the bend. The efficiency of this type of weld is very variable and its maximum value is in the region of 70 per cent., but 100 per cent. efficiency is now obtainable by electric arc welding. Many designers eliminate this weld and substitute for it the mitred corner of Fig. 127, in conjunction with the $6'' \times 4'' \times \frac{1}{2}''$ angle cleat. The expenditure of 11 lb. of metal in this cleat is more than paid for in the saving of the weld. From the remarks in the succeeding article it follows that the vertical leg of the $6'' \times 4'' \times \frac{1}{2}''$ angle cleat should have more rivets in it than are in the horizontal leg, the former rivets being in tension and the latter in single shear.

The Permissible Axial Tensile Stress on Rivets and Bolts is difficult to ascertain due to the high and variable initial tensile stresses set up when the newly-clenched rivet cools, and to the unknown initial screwing up stresses in the case of bolts, as was emphasized in Chapter IV. It has been previously stated that some engineers will not permit a rivet to carry an axial tensile load, but cases do arise in practice where rivets in tension can hardly be avoided. By using a low working stress for these rivets and subjecting them to rigid inspection as to soundness and fullness of head, quite efficient and trustworthy joints can be obtained.

The tables of Working Stresses on pages xiv and xv state that if the permissible tensile stress on net section is $F_t = 9$ tons per sq. in. then the working tensile stress on all shop rivets and on bolts less than $\frac{3}{4}$ -in. dia. is 5 tons per sq. in., and 4 tons per sq. in. on all site rivets. On bolts of $\frac{3}{4}$ -in. dia. and up the working stress is increased to 6 tons per sq. in. The net area of a bolt is that at the bottom of the thread.

To ensure an equitable distribution of the load among the tensile rivets in angles *B* and *C*, Fig. 125, it is suggested that these bracket angles should have a minimum thickness equal to two-thirds the rivet diameter less $\frac{1}{16}$ in., i.e., $\frac{2}{3}$ of $\frac{3}{4}'' - \frac{1}{16}'' = \frac{7}{16}''$ in the present example. When there is only one angle connecting the bracket plate to the column face the tensile loads on the rivets through the column flange are no longer axial, because of the eccentricity of the applied load, and the thickness of the angle should be at least two-thirds the rivet diameter plus $\frac{1}{16}$ in.

Fig. 128 is a riveted bracket wherein an attempt is made to minimize the tensile loads on the top rivets. A bracket of this type can be made much shallower than that of Fig. 125.

Figs. 129 to 132. In the first of these figures the bracket undoubtedly tends to heel about the bottom pair of bolts, and some text-books assume that the bracket of Fig. 125 does the same. When finding the resisting moment of rows *B* and *C*, they take the centre of rotation as being on the line *YY* at the bottom of the vertical angles, and the summation of the distances² as $2(1.5^2 + 4.5^2 + 7.5^2 + \dots + 28.5^2 + 31.5^2)$, *i.e.*, all the rivets in rows *B* and *C* are assumed to be subjected to axial tension. This method results in a very shallow bracket, and must seriously overstress the tensile fastenings. There is no reason for assuming the bracket to heel at the bottom. If there was an absolutely rigid projection from the column at the line *YY*, then something might be said for this method. When considering the plate girder web splice for

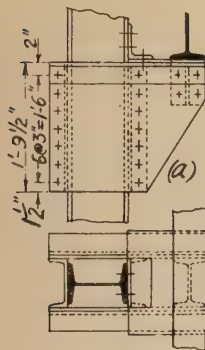


FIG. 128

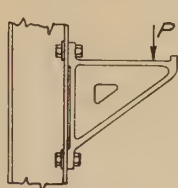


FIG. 129



FIG. 130



FIG. 131



FIG. 132

bending, rotation was not assumed at the top or compression edge *O* (Fig. 131), but at the neutral axis *N.A.* Similarly, with a cantilevered girder (Fig. 132), the web splice would not be calculated on the assumption that the cantilevered nose heeled about the compression edge at *O*, and, carrying the reasoning one step further, this is really the case of the bracket.

The centre of rotation is somewhere between the *XX* line and the *YY* line of Fig. 125. One authority assumes this neutral axis to be $\frac{1}{7}$ of the total bracket depth above the heel, *i.e.*, $\frac{1}{7}$ of 2ft. 9 $\frac{1}{2}$ in. or 4.8 in., approximately, above line *YY* on Fig. 125. The area of metal in direct contact below the *N.A.*, *viz.* 4.8 in. deep by 7 $\frac{5}{8}$ in. wide, is assumed to take the direct compression, while the fastenings above the *N.A.* are in axial tension. The tensile load on any rivet (or bolt) is directly proportional to its distance from the neutral axis, as previously explained. The vertical shear is, of course, shared equally by all the fastenings in rows *B* and *C*.

The method of calculation outlined for Fig. 125 is, therefore, on the safe side and was purposely adopted in view of the unknown value of the initial tensile stresses in the top fastenings of rows *B* and *C*.

CLEAT CONNECTIONS. Fig. 133

Lists of "standard" cleat connections are given in the "section books" of various firms. These should not be slavishly copied as being the proper connection for the joists written against them, but rather should be looked upon as suggestions or guides and nothing more. The diagram under discussion is a "standard" cleat for a 14" $\times$ 6" $\times$ 46 lb. R.S.J., web thickness = 0.4 in. On a 20 ft. 6 in. span the total uniformly distributed load necessary to cause an extreme fibre stress of 10 tons/square inch at mid-span is 20.5 tons; see p. xiv for permissible stresses. With this loading the end shear is 10.25 tons, while the shear stress on the web is $10.25^T \div$ web area of 14" $\times$ 0.4" = 1.83^T/sq. in. against the permissible stress of 6.5^T. The same joist carrying a uniformly distributed load of 72.8 tons on a 5 ft. 10 in. span has the maximum transverse fibre stress at mid-span of 10^T/sq. in., while the end shear stress on the web is $36.4^T \div (14" \times 0.4") = 6.5^T$ /sq. in., *i.e.*, both the permissible transverse and shear stresses are reached simultaneously. It would therefore be absurd to use the same cleat for a 14-in. R.S.J. for all spans and loading, as in the first example the end shear to be carried was 10.25^T, and in the second it was 36.4^T.

Loads on the Rivets in the Web of the 14-in. Joist. Span of joist 20 ft. 6 in.; end reaction, as in the first example, 10.25^T, Fig. 133.

The centre of gravity of the four rivets *A* to *D* is, by inspection, at *X*. Had the rivets been unevenly spaced, then the centre of gravity would be found by taking moments about a horizontal and a vertical line of reference.

Bending moment about *X* = $P.e = 10.25^T \times 3\frac{3}{8}" =$ in. tons 34.6

Σ distances² = $XA^2 + XB^2$, etc., = $4XA^2$

= $4 \times 3.204^2 =$ in.² 41.1

Load *L* on each rivet due to *B.M.* = $(P.e \times$
extreme radial length) $\div \Sigma$ dist.² (and acts
clockwise and at right angles to each radial
line) = $34.6 \times 3.204 \div 41.1 =$ tons 2.7

Direct vertical load per rivet = $P \div$ number of
rivets = $10.25^T \div 4 =$,, 2.56

Resulting load on the *A* and *B* rivets, acute angle
between the forces = ,, 4.35

Resulting load on the *C* and *D* rivets, obtuse angle
between the forces = tons 2.99

The rivets are $\frac{3}{4}$ in. diameter and from p. xv, the *D.S.* value per rivet = 5.3^T and the bearing value on the 0.4-in. web is 3.6^T , from which it would appear that the *A* and *B* rivets are seriously overstressed to the extent of 21 per cent.

The above treatment is given in text-books without further remark, and it would appear at first sight that the quite common practice of designing cleat connections for the direct load only—i.e., neglecting the bending moment of $P.e$ —is greatly in error. However, there is one important assumption made in the foregoing calculations, viz., that the bolts *E, F, G, H, I* and *J* are in single shear only. On this basis the thrust from the beam causes a downward load on each bolt of $10.25^T \div 6 = 1.71^T$.

In practice the deflection of the 14-in. joist is small, and thus the

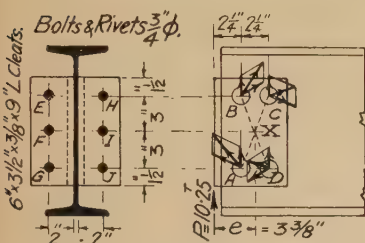


FIG. 133

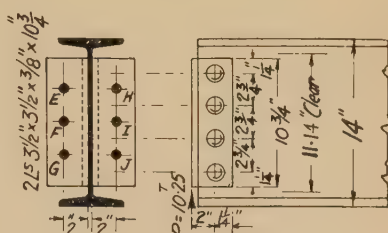


FIG. 134

slope of the joist at its ends must also be small, but, however small it may be, it will always create axial tension in the upper bolts *E* and *H*. These must elongate a trifle under stress, and so the cleat faces *E* and *H* also tend to rotate as well as to move vertically downwards.

Again, if *E* to *J* were rivets, and *A* to *D* were bolts with $\frac{1}{16}$ in. clearance in the holes and with the nuts not screwed too tightly up against the cleat faces (i.e., loose pins), then the load on the four bolts *A* to *D* would be central through *X*, since the joist is free to deflect a small amount. The eccentricity of *e* still exists, but now causes a *B.M.* of $P.e$ on the rivets *E* to *J*. This connection, however, is not used in practice, as it entails the threading of a heavy joist between two cleats.

Any simple practical calculation for strength must necessarily be based upon some questionable assumption. Thus the following have all a bearing upon the load distribution:—Faces not absolutely flat; clearance of bolts in holes; different degrees of tightness of nuts and rivet heads against the cleat faces; relative rigidity of the joist and the parent structure; initial axial stress in the rivets and

bolts ; frictional resistances of faces in contact. See the writings of Marshall, Hoo and Francis (end of Chapter References).

Some attempt has been made to standardize the calculation of cleat connections, but it must be admitted that many designers when engaged upon beam work simply design the connection for direct load only, and this with no apparent ill effects.

Fig. 134. With the knowledge that eccentricity stresses do exist a satisfactory cleated connection can be designed which will be quite in keeping with the stricter theoretical investigation given above by:—(1) Reducing the lever arm e as much as possible, and (2) giving the number of rivets slightly in excess of the direct load requirements. Table 5 (Appendix) states that the maximum clear depth between the root fillets of a 14-in. joist is 11.14 in., thus allowing the use of the 10 $\frac{3}{4}$ -in. deep cleat. The eccentricity e can be reduced to 2 in. by using 3 $\frac{1}{2}$ " $\times$ 3 $\frac{1}{2}$ " $\times$ $\frac{3}{8}$ " angles, Fig. 134.

Designing for Direct Load only.

Number of rivets required in joist web =	
10.25 $\tau \div 3.6\tau$ i.e., the bear. value/rivet =	2.84
Number of rivets given =	4

Now Check for Bending Stresses.

$\Sigma \text{ dist.}^2 = 2 (1\frac{3}{8}^2 + 4\frac{1}{8}^2)$	= in. ²	37.82
$P.e = 10.25\tau \times 2"$ on the rivets A to D	= in. tons	20.5
Load L on the rivets A and D , acting horizontally, and clockwise about $X = 20.5 \times$ extreme lever arm $\div \Sigma \text{ dist.}^2 = 20.5 \times$ $4\frac{1}{8} \div 37.82$	= tons	2.23
Direct vertical load per rivet = $10.25 \div 4$	= „	2.56
Resultant load on the rivets A and D = $\sqrt{(2.23^2 + 2.56^2)}$	= „	3.39
Which is less than the bearing value per rivet of	„	3.6

Fig. 135. A case where bending moment is constantly neglected is that of the simple angle and gusset plate connection. The force P , which in this example is supposed to act along the centre of gravity line of the angle, has its reaction along the mid-line of the gusset plate. These two equal and opposite forces being at a distance e_1 apart, form a couple. The wider the non-riveted leg of the angle the greater will be the distance of the centre of gravity line from the heel, and, therefore, the larger the couple. The eccentricity e_1 will have a minimum value when the non-riveted leg shrinks to zero, i.e., when the angle becomes a flat bar.

If the three rivets through the angle were just sufficient to develop the direct stress in the angle, then they must be overstressed due to

the additional bending stresses caused by the eccentricity. However, in the case of a single angle with end gusset connection the eccentricity is relatively a small quantity (on an average about 1 in.) and the resulting bending moment is neglected. The connection is thus designed for direct stress only, with the rivets in shear and bearing.

In good practice a minimum of two rivets is generally used in a connection of this kind, partly to counteract the above eccentricity stresses, and partly from the principle that if one rivet was used it might be faulty, possibly with serious results; the probability of both rivets being bad is remote. If one rivet in a joint composed of several rivets is faulty then its load will be shared among the remaining rivets, each of which will now carry a slightly higher load than that originally assumed. Owing to the eccentricity which exists in a joint of this type the number of rivets given should be, preferably, on the full side.

Fig. 136. The long leg of this unequal angle is riveted at both ends, and the load acting on the angle has a tendency to remain in the riveted leg, since the load is given to this leg by the rivets. The riveted leg will elongate under load, but can only do so if it takes the unriveted or outstanding leg with it. Since stress is proportional to strain, the outstanding leg, therefore, will be stressed also. The theoretical distribution of stress throughout the cross-section of the angle can be found as in the numerical example which follows. Considering the number of simple tension members used in practice, this method is too tedious and an approximation is used in its stead. The approximate method virtually assumes that the intensity of stress is constant across the effective net area, and equals $P \div \text{effective area}$.

By most specifications the effective net area is assumed to be that uncoloured in the figure, and equals the area of the riveted leg minus the hole, plus half the area of the outstanding leg.

Note that the outstanding leg is measured from the root of the angle to the toe and not from the heel to the toe.

The foregoing rule is universally employed in specifications for (equal and) unequal angles when the long leg is the riveted one. The same rule is employed by some specifications in the unusual case of an angle tie having rivets through the short leg. The B.S. 449 states that when the short leg is the riveted one then the effective area of an angle tie is the net area of the riveted leg plus half the gross area of the riveted leg. Tests would indicate this rule to be uneconomical in that the angle has a larger effective area (and therefore a larger capacity for carrying load) than that suggested by this particular specification.

When an angle has the end connecting rivets through only one

leg, then it follows from the above and the text to Fig. 135 that it is more economical to employ an unequal angle with the rivets through the long leg.

Numerical Example, Fig. 136

Effective Area Method. The tables, to which references are made, are those given in the Appendix. Tension is given negative sign.

Gross area of $5" \times 3" \times \frac{3}{8}"$ L, Table 2, sq. in. 2.86

Less hole, $\frac{3}{4}" \times \frac{3}{8}"$ = 0.28 „ 9

„ $\frac{1}{2}$ leg, $\frac{1}{2}(3" - \frac{3}{8})\frac{3}{8}"$ = 0.49 „ 10

Or, as in Table 10, = 0.77 „ 0.77

Effective area = „ 2.09

Stress intensity = $-10^7 \div 2.09$ = $\tau/\text{sq. in.} -4.78$

Taking bending into account at section $\beta\beta$

Gross I_a Table 2, in.⁴ 7.25

Gross Z , top = $7.25 \div 1.68$ = „ in.³ 4.31

„ „ bot. = $7.25 \div 3.32$ = „ „ 2.18

B.M. = Pe_2 = $10^7 \times 1.07$ = in.⁷ 10.7

Extreme fibre stress due to B.M. at:—

$AB = M \div Z_t$ = $10.7 \div 4.31$ = $\tau/\text{sq. in.} +2.48$

$C = M \div Z_b$ = $10.7 \div 2.18$ = „ -5.00

Direct stress ($\beta\beta$) = $-10^7 \div 2.86$ = „ -3.50

Sum, AB, Fig. d = $+2.48 - 3.50$ = „ -1.02

Sum at C, Fig. d = $-5 - 3.50$ = „ -8.50

Average stress intensity = $-\frac{1}{2}(1.02 + 8.5)$ = „ -4.76

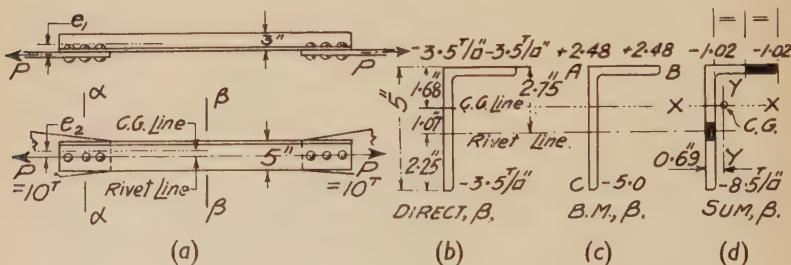


FIG. 135

FIG. 136

The procedure is the same when section $\alpha\alpha$ is considered except that the net area of the holed angle must be used. Thus, the net area of the holed angle is the gross area less one rivet hole only. Next, by moments, find the new centre of gravity line XX of the holed angle. (The new y_t and y_b will no longer be 1.68 in. and

3.22 in., respectively.) Calculate the net moment of inertia of the angle with one hole and thus obtain the net Z_t and net Z_b .

Fig. 137. Tension Members. In certain lattice girders the web diagonals are in tension and are subject to the foregoing rule. The bottom boom is also in tension, but is not made to suffer the same sacrifice of metal. In the case of these flange angles the effective area is generally assumed to be the gross area of the angles minus the rivet holes only. The stress in the bottom flange increases from zero at the end panels to a maximum in the centre panels. The pull in these angles has been increased gradually, and the stress has had time, as it were, to permeate to the outermost fibre of the angles. Thus at the girder centre line, where the stress has reached its maximum intensity, the angles of the bottom flange are assumed to be evenly stressed throughout their cross-sectional area. The

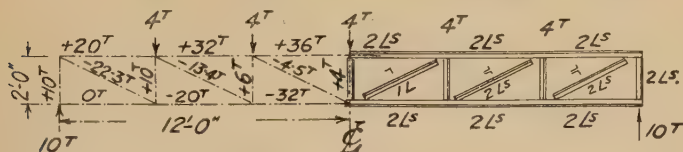


FIG. 137

diagonals, on the other hand, had their loads given to them instantaneously by the end gussets.

Compression Members are assumed to be effective in carrying stress throughout their full cross-sectional area with no deductions for the outstanding leg. In the case of a strut the further the metal is away from the centre line the greater is the radius of gyration and the stronger is the member. A hollow shaft is stronger as a column than a solid shaft, where both columns have the same cross-sectional area of metal.

Tension members used to be made always of flat bars, but modern practice uses angles or built-up sections where possible. The angle gives a greater rigidity to the structure than the flat bar, and rigidity is of as great importance as strength. In many light bridges flat bar tension members can be seen to vibrate during the passage of the load, while the compression members, being necessarily of a section other than that of a flat bar, can hardly be observed to vibrate at all. Again, the girder may have to act either intentionally or unintentionally as part of a portal frame, and there is thus the possibility of a stress reversal in some members. Angles can take this reversal from a tensile to a compressive stress, whereas flat bars are unfitted to carry a compressive load due to their small radius of gyration in one direction.

FRAME CENTRE LINES

The skeleton frame of the truss of Fig. 137, formed by the "centre lines," is shown on the left hand and the completed truss minus gusset plates is indicated on the right. The "centre lines" may be either the rivet lines of the angles or the centre of gravity lines. Since these two sets of lines are not coincident, there is always an eccentricity of loading either on the rivets or on the bars. Roughly the practice is :—(1) Where only angles are used the skeleton frame is composed of the rivet lines, and in the case of angles with double rivet lines the "centre lines" are the rivet lines which are nearer the heels.

(2) Where compound sections are used the "centre line" is the centre of gravity line of the built-up section. These centre lines are laid down on the paper first and the angles or compound sections placed in their relative positions afterwards.

Eccentric Frame Centre Lines. It is axiomatic that the centre lines, whether gravity or rivet lines, should intersect at the panel points. This should be carried out in practice, although there are cases where this rule may be relaxed, particularly in light structures, but only after due regard has been paid to the stresses involved.

Fig. 138 is a common attempt to economize on gusset plates, which are absolutely necessary, as the correct detail of Fig. 139

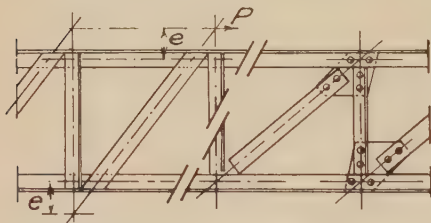


FIG. 138

FIG. 139



FIG. 140

indicates. The bending moment $P \cdot e$ is shared by the members directly as their moments of inertia and inversely as their lengths. P is the increment of flange force at the panel point, *i.e.*, the difference of the forces in the portions of the flange to the left and right of the panel point. If I = moment of inertia and L = the length of a bar, then the proportion of the bending moment carried

$$\text{by a bar } A = \left(\frac{P \cdot e}{\sum \frac{I_A}{L_A} + \frac{I_B}{L_B} + \frac{I_C}{L_C} \dots} \right) \times \frac{I_A}{L_A}$$

The total stress in bar *A* is the direct or primary stress added to the stresses due to bending or secondary stresses.

Fig. 141 is the mid-rafter detail of Fig. 140 and has the simplest gusset plate possible. The number of rivets through the gusset

A = 2L³ 3" x 2½" x ¼" *B* = 2L³ 2½" x 2½" x ¼" *C* = 1L 2½" x 2½" x ¼" *D* (Fig 144) 1L only.

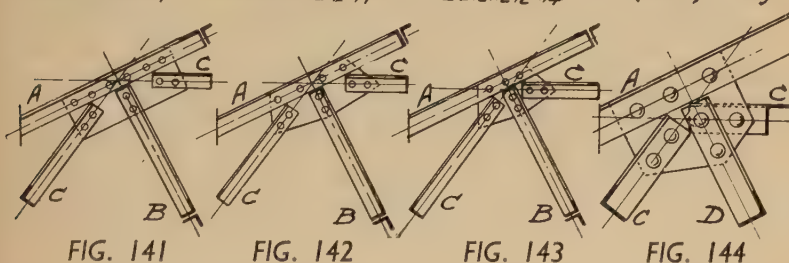


plate and rafter angles *A* is generally small, and extra rivets are given for the purpose of stitching the elements together. These are pitched at 12*t* (or 16*t*, see text to Fig. 44) apart.

Fig. 142 is an effort to lighten the appearance of the joint and, at the same time, to economize in weight of material and riveting, but the latter is only gained at the expense of the labour expended upon the plate. Too sharp a corner formed in the gusset plate where it overlies the bars *C* should be guarded against, as tapered corners spring away from the more rigid section. Both the present and the previous details satisfy the principle of intersection of centre lines.

Fig. 143. The gusset plate is here reduced to a minimum consonant with symmetry, but only by introducing secondary stresses caused by the couple *P . e*, as in Fig. 138.

Fig. 144, drawn to twice the scale of its neighbours, exhibits still another fairly common attempt to cut down gusset area and riveting. Frame centre lines are eccentric, and so also are the rivet groups. False economy with gusset plates may easily result in a dangerous structure.

Fig. 145 illustrates the one time almost universal British detail for a roof truss shoe. The members have secondary stresses, because of the eccentricity of the centre lines, in addition to the primary stresses, while the rivets carry a direct load plus a load due to bending moment.

The bending moment = $V \cdot ev = H \cdot eh$, because $\frac{H}{R} = \frac{ac}{ab}$ and

$$\frac{V}{R} = \frac{cb}{ab}, \text{ and so } V \cdot ev = R \frac{cb}{ab} \cdot ac \text{ and } H \cdot eh = R \frac{ac}{ab} \cdot cb.$$

A clearer conception of the bending action can be obtained from Figs. 147 and 148. In the former the tie is replaced by its pull

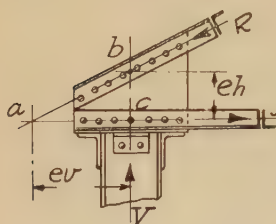


FIG. 145

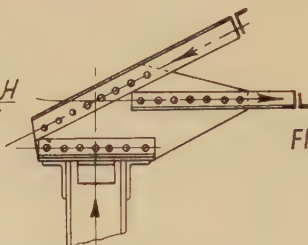


FIG. 146

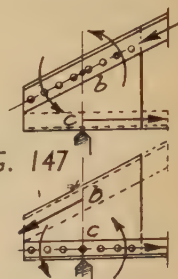


FIG. 148

acting from the rivet centre of gravity c , and in the latter the rafter is assumed to be non-existent and replaced by its thrust acting at the rivet centre of gravity b . The gusset plate, which is cantilevered out from the remaining angles, transfers the counter-clockwise couple into those angles through the rivets. The bending moment is therefore shared by both rafter and main tie through their rivets, and, assuming a knife-edge support at the column cap under c , the bending moment on the rafter would be

$$\frac{I_R}{L_R} \times \frac{V \cdot ev}{\frac{I_R}{L_R} + \frac{I_T}{L_T}};$$

the subscripts $_R$ and $_T$ denoting rafter and tie respectively. There is, however, a large counteracting moment offered by the rivets through the cap cleats which greatly reduces the bending moment on both rafter and tie.

Fig. 146. This detail eliminates the foregoing secondary stresses, but is more used in America than here. It is not always advantageous, however, to use this detail. With wind bracing at roof tie level the wind load must get into the column shaft and so to the ground. In that detail more common to this country the shoe horizontal base plate is extended into the shop and also acts as a gusset plate for the horizontal wind-bracing system. The load is thereupon transferred into the column without passing through a raised and slender vertical plate. This point is reverted to in Chapter VI., Vol. II.

If the vertical gusset plate in Fig. 145 be made $\frac{1}{16}$ in. or $\frac{1}{8}$ in. thicker than the other gusset plates of the truss and if the rivets be given slightly in excess, as previously advocated, then there is no

objection to the use of this type of shoe, in fact this shoe is preferred by many as being more efficient and rigid.

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CHAPTER IX

WIND PRESSURE—FACTOR OF SAFETY

"THE vagaries of the wind" is an expression used lightly by many, but not by those engaged upon the competitive design of high structures, to whom the term is pregnant with meaning. The wind is so erratic in behaviour that it is impossible to foretell the actual maximum wind pressure which will act upon a structure. A structure can be designed to withstand any wind which may blow, but economy demands that a structure need not withstand a tornado when there is no possibility of such an occurrence.

Pressure and Velocity have been connected together by the formula $p = kv^2$. The value for constant k for the pressure on a flat plate placed normal to the wind is 0.0031 and is obtained as follows.

In a fluid with steady stream line flow, the sum of the kinetic energy (velocity head), the pressure energy (pressure head) and the potential energy (elevation head) is constant—Bernoulli's theorem (A.D. 1738).

If a stream line in a fluid be brought to rest by making it impinge on a tiny flat plate, normal to the flow, then the pressure acting on the small plate will be equal to the total head at that point before the plate was introduced, on the supposition, of course, that the one stream line only is affected.

For a wide area of wind flow a flat plate 1 ft. square is relatively a small plate. If the free wind stream is horizontal then the only energy it possesses is kinetic energy equal to $\frac{1}{2}wV^2/g$. Insertion of the plate into the wind brings one particular stream line to rest and, by Bernoulli's theorem, the pressure on the plate must equal the kinetic energy in the unimpeded stream before it was brought to rest.

Let w = weight in lb. per cub. ft. of air, = 0.0765.

v = wind velocity in miles per hour.

V = wind velocity in ft. per sec.

$= v \times 5,280 \div (60 \times 60) = (22/15)v$.

$g = 32.2 \text{ ft./sec.}^2$

$$\begin{aligned}
 \text{Hence } p &= \frac{1}{2} \times \frac{w \text{ lb.}}{\text{ft.}^3} \times \left(\frac{V \text{ ft.}}{\text{sec.}} \right)^2 \times \frac{\text{sec.}^2}{g \text{ ft.}} = \frac{1}{2} \frac{w V^2 \text{ lb.}}{g \text{ ft.}^2} \\
 &= \frac{1}{2} \times \frac{0.0765}{32.2} [(22/15)v]^2 = \left(\frac{1}{2} \right) 0.00511 v^2 \quad \dots \quad 1(a) \\
 &= 0.00256 v^2 \quad \dots \quad 1(b)
 \end{aligned}$$

In deriving this theoretical formula no allowance was made for viscosity and it was assumed that all the air in the stream line impinged upon the plate and was duly brought to rest and, also, all this occurred without interference with the surrounding air stream lines. Obviously, therefore, this formula must be corrected experimentally, and it has been found that if the theoretical coefficient $\frac{1}{2}$, shown within brackets in 1(a), be replaced by the experimental coefficient 0.6 a more correct value of the pressure is obtained. Hence on a flat vertical surface normal to a horizontal wind the pressure

$$p = 0.6 w V^2 \div g = 0.6 \times 0.00511 v^2 = 0.0031 v^2 \quad \dots \quad 1(c)$$

in lb. per sq. ft. when v is in miles per hour.

Velocity and Height above Ground. The pioneering records of storm pressures at the Forth Bridge (Arrol's handbook) have now

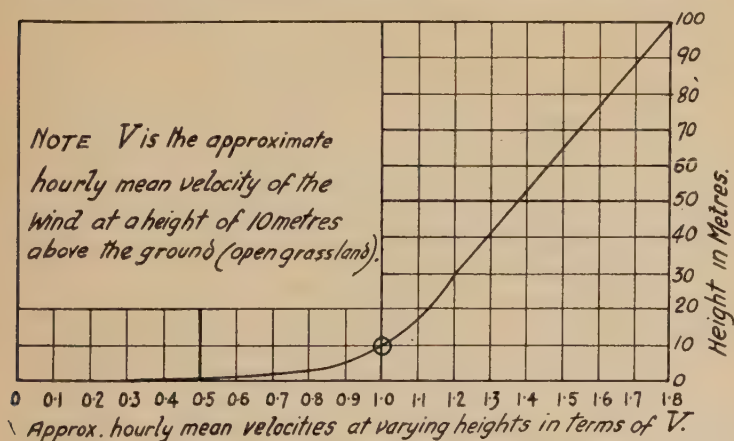


FIG. 149

been found to be too high through errors due to inertia, nevertheless, the ratio existing between the readings will remain fairly constant although the absolute values may be somewhat incorrect. In the following table the average pressure for a period of several years is p in pounds per square foot on the gauge at the lowest level.

and Hutton formulæ. No allowance was made for the suction or negative pressure on the leeward side of a structure. These formulæ are gradually falling into disuse in Europe, but they are still employed in the United States (especially the Duchemin formula), where many building codes make no provision for suction.

Let p be the horizontal pressure in pounds per square foot of vertical surface, and p_n the resulting normal pressure, also in pounds per square foot, then :—

$$(1) \quad p_n = p \frac{2 \sin \theta}{1 + \sin^2 \theta} \quad . \quad . \quad . \quad (\text{Duchemin.})$$

(2) $p_n = p \frac{\theta^\circ}{45^\circ}$, where θ is not greater than 45° . (Straight line.)

$$(3) \quad p_n = p \sin \theta^{1.84 \cos \theta - 1} \dots \dots \dots (\text{Hutton.})$$

Modern investigations have clearly demonstrated that the wind force acting on a structure depends more upon the shape of the leeward side of the obstruction, or structure, than on the shape of the windward surface presented to the wind. The resistance offered by an

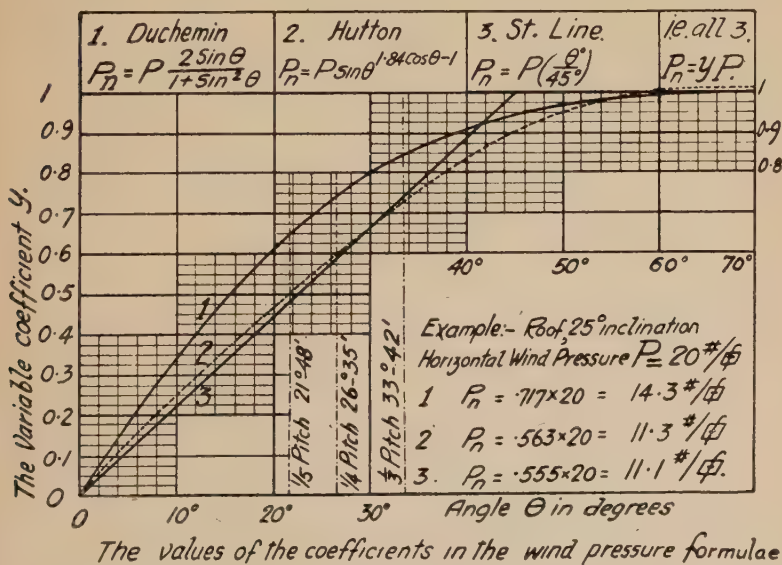
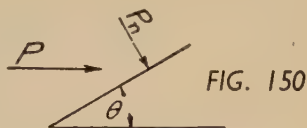


FIG. 151

is due to the differences of pressure and to the friction caused by the wind flowing round the obstacle in its path.

The relationship between pressure and shape* is mentioned in British Standard 449 in its "Table 6" quoted under.

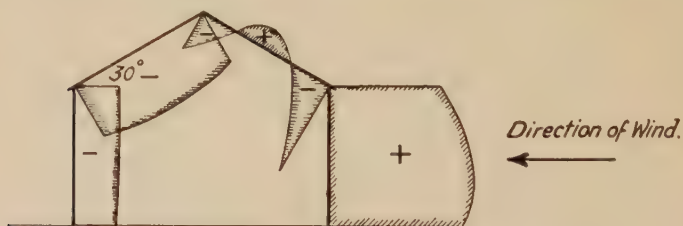
TABLE 6. B.S. 449

Intensity of unit wind pressure (p) on projected area in wind direction.

Plan Section of Projection.	Ratio of Height of the Projection to Base-width of the Projection.		
	0 to 4	4 to 8	8 to 16
Circular	$0.7p$	$0.7p$	$0.7p$
Octagonal	$0.8p$	$0.9p$	$1.0p$
Square (wind perpendicular to diagonal)	$0.8p$	$0.9p$	$1.0p$
Square (wind perpendicular to face)	$1.0p$	$1.15p$	$1.3p$

The value of p is that obtained from wind formula (2) where h is the height measured to the top of the projection.

Vertical Walls. Wind tunnel experiments have contributed greatly to our knowledge of wind pressure. Fig. 152 roughly indicates the distribution of the wind pressure on a model of a building when placed in the path of a wind in the tunnel at a uniform velocity of about 17 m.p.h. (Dr. Stanton, *Min. Proc. Inst. C. E.*, Vol. CLVI.).



Normal Wind Pressures on a Model of a Building.

FIG. 152

The positive pressures are plotted on the outside of the building and the negative pressures inside.

* That there was a relationship between pressure and shape was known to the Aztecs long before Sir Isaac Newton connected p with v^2 . The Aztec temples of the god of wind, *Ehecatl*, were the only temples built in cylindrical form.

When the wind blows at right angles to a building the B.S. 449—Building, says that for the purposes of design the external pressure normal to the windward wall shall be taken at $0.5p$ (right to left, Fig. 152) and the external suction, or negative pressure, on the leeward wall (also from right to left) at the same value of $0.5p$, where p is obtained from wind formula (2), with h measured from the ground to the ridge of the roof. (In Holland the values are $0.9p$ and $0.4p$, respectively.)

Windward Roof Slopes. The same specification classifies all surfaces inclined at 70° or more to the horizontal as being equivalent to vertical surfaces. As the angle of inclination to the horizontal decreases so also does the positive normal wind pressure decrease. When the inclination is 30° the resulting normal pressure is zero : below 30° a suction is specified as acting outwards normal to the slope. This suction increases in value as the angle decreases towards zero.

θ varying from	Pressure intensity, normal to roof.		
0° to 30°	$p(\theta/30 - 1)$	3(a)	
30° to 45°	$p(\theta/60 - 0.5)$	3(b)	
45° to 70° (and over) . .	$p(\theta/100 - 0.2)$	3(c)	

Leeward Roof Slopes at all angles of inclination to the horizontal are specified as being subjected to an outward acting force, or suction, of the constant value of $0.5p$ (in Holland the value is $0.4p$).

Flat Roofs. On the windward half of a flat roof, since θ is zero, the suction by formula 3(a) is $-p$ and on the leeward half it is only $-0.5p$. (In Holland the value is $-0.4p$ overall the flat roof.)

Summary. The following table shows the variation of the pressure intensity with the angle θ for windward (formula 3) and leeward slopes.

Slope of Roof, θ° .	External Pressure.		Slope of Roof, θ° .	External Pressure.	
	Windward.	Leeward.		Windward.	Leeward.
0° . . .	$-p$	$-0.5p$	30° . . .	0	$-0.5p$
10° . . .	$-0.67p$	$-0.5p$	40° . . .	$+0.17p$	$-0.5p$
20° . . .	$-0.33p$	$-0.5p$	45° . . .	$+0.25p$	$-0.5p$
21.8° ; rise, $\frac{1}{2}$ span	$-0.273p$	$-0.5p$	50° . . .	$+0.3p$	$-0.5p$
26.57° ; rise, $\frac{1}{2}$ span	$-0.114p$	$-0.5p$	60° . . .	$+0.4p$	$-0.5p$
			70° and upwards	$+0.5p$	$-0.5p$

Flat roof :—Windward half = $-p$; leeward half = $-0.5p$.

Velocity and Locality. The configuration of the ground and

adjoining structures both influence the wind velocity and, therefore, the pressure. A bridge which spans a deep funnel-shaped gorge may be expected to experience a higher wind pressure than one which bridges a placid stream in a grassy plain, while a low building entirely surrounded and screened by high warehouses may never be subjected to any wind pressure worth mentioning.

Similarly the maximum recorded wind velocity (and hence pressure) varies from country to country. Even in the British Isles there is quite a difference between the intensities of storms experienced in Central England and in the West of Scotland. This fact is embodied in the B.S. 449—Building, which gives different values for velocity v as follows.

Central England (including the Severn estuary). Considering this, the most sheltered part of the British Isles, the following values for v are to be used in the design :—

Where the building will be

- | | |
|---|-----------------|
| (a) Shielded from the full effects of the wind by the configuration of the ground | $v = 50$ m.p.h. |
| (b) Situated in open and comparatively flat country not higher than 500 ft. above sea level | $v = 60$ m.p.h. |
| (c) Very exposed and on a site at least 900 ft. above sea level | $v = 70$ m.p.h. |

East Scotland, N.E., E., S.E. and S. England and N.E. Ireland.
The above values are increased to 55, 65 and 75 m.p.h., respectively.

West Scotland, Wales, N.W. and S.W. England, also N.W. Ireland.
The values for v are increased by 10 m.p.h. to 60, 70 and 80 m.p.h. respectively for positions (a), (b) and (c).

Reference may be made to Gould (Presidential Address, Royal Meteorological Society, 1936) and to the joint paper by Bailey and Vincent (Inst. C.E. 1943) for a comprehensive list of the maximum wind velocities as measured at a large number of stations in the British Isles. The accompanying table is an abstract arranged in the same order as the table of design values given above.

The value of the wind gust-velocity recorded depends upon the height at which the Dines anemograph was installed. By means of a wind-gradient graph, similar in nature to that of Fig. 149, the velocity values in the table are adjusted values arranged to read as if all the instruments had been installed under standard conditions at an effective height of 40 ft. above open grass land. Thus the storm of January 28th, 1927, the worst so far recorded in Edinburgh and one which caused considerable damage, was recorded as 85 m.p.h., but adjusted to standard conditions the velocity is 94 m.p.h., equivalent to a pressure of 0.0031×94^2 or 27.4 lb. per sq. ft.

Previous to the use of instruments for measuring wind velocities observers classified storms by the damage done. Thus, Admiral Beaufort (1805), in his famous but now obsolete scale for seamen, stated that a whole gale was one which was "seldom experienced inland ; trees uprooted ; considerable structural damage done." Then later on the wind pressure was ascertained by calculation after examination of the damage done ; knowing the section and material of a broken member and the total area of structure presented to the wind, it is possible to find the total wind load, and hence the average intensity of the pressure, which caused the failure.

TABLE OF MAXIMUM GUST-VELOCITIES

(From Bailey and Vincent)

Position.	Station.	Observation Period, Years.	Velocity at 40 ft. Effective Height, m.p.h.
CENTRAL ENGLAND .	Birmingham	12	69
EAST SCOTLAND .	Balmakewan (Aberdeen) .	20	95
	Edinburgh	21	94
N.E. & E. ENGLAND .	Spurn Head (Yorks) . . .	14	86
	Cranwell (Lincs)	12	83
	Gorleston (Norfolk) . . .	25	80
S.E. & S. ENGLAND .	Shoeburyness (Essex) . . .	24	71
	Kew (London)	30	69
	Dover	11	77
	Larkhill (Salisbury Plain) .	6	77
N.E. IRELAND .	Aldergrove (Belfast) . . .	8	97
WEST SCOTLAND .	Paisley (Renfrew)	22	108
WALES	Holyhead	30	87
	St. Ann's Head (Pembroke) .	1	100
N.W. ENGLAND .	Fleetwood (Lancs)	10	88
	Liverpool	7	91
S.W. ENGLAND .	Pendennis Castle (Cornwall) .	23	101
N.W. IRELAND .	Dunfanaghy (Donegal) . . .	7	99
W. IRELAND	Quilty (Co. Clair)	25	>116

Sir Benjamin Baker's experiments with a glazed window as the bottom of a water tank led him to believe that ordinary windows would break at a pressure somewhere in the neighbourhood of a pressure of 30 lb. per square foot.

Coming nearer the ground, one seldom finds it reported that a tram-car has been blown over. Thus if p is the pressure in pounds per square foot on the car, then (Fig. 153) $p \times \text{area} \times \text{lever arm}$ to the centre of pressure = stabilizing moment of $W \times \text{half the gauge}$.

$p \times 432 \times 8\frac{1}{3} \text{ ft.} = 13^T \times 2.375 \text{ ft.}$, whence $p = 19 \text{ lb. per square foot}$.

With the car empty the required pressure for overturning is only 15 lb. per square foot.

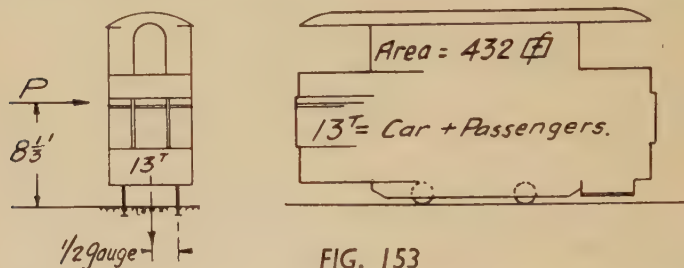


FIG. 153

It may be assumed, therefore, that the maximum gust velocity of the wind in Great Britain is in the region of 100 m.p.h., giving a maximum pressure of roughly 30 lb. per square foot.

In the absence of an anemometer it is possible to form an approximate estimate of the velocity of the wind from observations of the movements of the leaves and branches of trees, as given in the following table.

Wind Velocity in m.p.h.	Description.	Action on Trees.
0—2	Calm	Leaves still.
5	Gentle breeze . .	Leaves just move.
10	Pleasant breeze .	Branches just move.
15	Strong breeze . .	Branches just bend.
20	Moderate wind . .	Branches sway and swing violently.
25	Strong wind . .	Leaves torn from trees.
35	Gale	Light branches broken.
55	Storm	Strong branches broken.

Maximal Velocities and Pressures in Other Countries. In the north of the United States, Canada and Australia the horizontal wind pressure is taken at 30 lb. per square foot.

In the south of the United States and some parts of India fierce tornadoes or cyclones frequently occur. At the meeting of the American Association for the Advancement of Science in Phila-

delphia it was stated that a new record was set up by the hurricane at Miami on September 18th, 1926. At 7.40 a.m. the wind blew with a velocity of 132 m.p.h., and thirty-two minutes later the wind gauge was blown down. This velocity is equivalent to a pressure of 54 lb. per square foot. The amount of damage done by this storm was phenomenal.

It will be recalled, however, that tornado damage is caused not only by the very high wind velocity but also by the low pressure in the central core.

On the subject of Mount Washington the *National Geographic Magazine* of America states that the previous record of wind velocity at the top of this mountain was broken on "April 12, 1934, when the anemometer registered about 200 miles per hour."

Gust and Average Pressures. The intensity of the wind pressure is seldom constant over a large area, for somewhere in the path of the wind will be found stream filaments or "pockets" of wind travelling at a higher velocity than the surrounding wind, and, therefore, exerting a higher pressure upon the gauge. On an average it would appear that the mean velocity is about 75 per cent. of the maximum gust velocity. The duration of the gust pressure is relatively small, somewhere about one or two seconds.

The large gauge of 300 sq. ft. at the Forth Bridge showed lower pressures than the small gauge of $1\frac{1}{2}$ sq. ft. which was inset in the larger gauge.

Very little research work has been carried out upon this particular aspect of wind pressure.

Wind-tunnel Experiments. It has become increasingly popular to make use of models in the design and construction of hydraulic works. In the design of long-span bridges recourse, with beneficial results, has been made to wind-tunnel experiments and it is now the general opinion that similar tests should be made for buildings, especially when the undertaking is a large one, or unusual in shape, or when sited on an exposed position.

Scale Effect. Although there still exists a difference of opinion it would appear that experimental results from models can be used directly, with negligible error, in the design of the prototype, provided that the structure is sharp edged throughout. There is, therefore, virtually no scale effect : this is not so when the structure has curved external surfaces.

Cost. No figures are available whereby the cost of the experimental work can be stated as a percentage on the cost of the finished work, even if such values were available they would be fallacious when applied as a general rule.

Internal Pressure in a building is caused by the wind blowing

through an open end or side. If the wind blows in the opposite direction a suction is created within the building. In a dwelling house no such large scale action is possible because of the internal partition walls. The B.S. 449—Building gives four values, *viz.*, $+0.2p$ and $-0.2p$ for ordinary permeability (*e.g.*, a shop covered with corrugated sheeting) and $+0.5p$ and $-0.5p$ when the building has large openings. Calculations involving internal pressure and suction are given in Chapters V and VI of Volume II. Some Continental specifications give, in great detail, values for buildings with different types of large openings.

Wind and Snow Loads. It is stated in the B.S. 449—Building that the combined effect of snow and wind shall be considered. The snow load is 10 lb. per sq. ft. of plan area on all roofs with slopes between 10° and 65° ; on roof slopes below 10° the allowance is a 2 ft. depth of loose snow, and on roofs inclined at more than 65° no snow at all. In some European specifications the simultaneous occurrence of maximum snow load and maximum wind pressure is not considered possible.

The Load Caused by Wind although dynamic in character is nevertheless considered as a static load, *i.e.*, there is no increment made for "impact" effect. In fact, in many specifications a higher working stress than usual is employed when dealing with wind loads. The reason is that the maximum wind pressure assumed in the design calculations can only occur very infrequently and a temporary overstress (*provided that the elastic limit is not exceeded*) is permissible for a second or two in a structure's lifetime of many many millions of seconds.

Wind on Bridges. Both Sir B. Baker and Sir J. Fowler recognized that the (Board of Trade) figure of 56 lb. per square foot of flat surfaces for wind pressure was too high for design purposes for the Forth Bridge. One must recall, however, the then recent Tay Bridge disaster and its reverberations upon the public mind, the immensity and daring of the project, together with the absolute lack of reliable wind data, before passing any comment upon the Report of the Commission.

Most of the world-famous bridges have been designed for a 30-lb. wind. This, in itself, means a large saving in weight, as can be seen from the fact that of the total estimated load on one of the cantilever bottom booms of the Forth Bridge the specified wind contributed 47 per cent.

In many English-speaking countries the custom in bridge design is to allow for a 50-lb. wind on an unoccupied bridge and a 30-lb. wind on both bridge and train when the bridge is occupied. The wind, moreover, is assumed to act as a live load (without impact

addition) at a slight angle to the bridge elevation, thereby blowing on both main girders. The intensity of pressure on the exposed leeward girder is : (a) the same as that on the windward girder if the two main girders are wider apart than twice their depth, and (b) one-half the standard pressure if less than this distance apart, thus allowing for the sheltering effect of the windward girder.

FACTOR OF SAFETY AND WORKING STRESS

Ultimate Strength of mild steel in pure tension is a variable quantity and may have any value between 28 and 33 tons per sq. inch.

Yield Point and the elastic limit are practically identical in the case of mild steel for structural work. The minimum value of the yield stress is about 15.25 tons per square inch, B.S. 15 : 1948.

Provided that the elastic limit is not exceeded, steel is an elastic material. In keeping with Hooke's Law (A.D. 1678) the elongation will be doubled if the force is doubled, and the steel will return to its original unstrained form on the withdrawal of the force. If the force be increased to such an extent that the resulting stress exceeds the yield stress then permanent distortion occurs. Thus a tie, or tension member, may be gradually loaded to about 30 tons per square inch before it breaks, but its practical usefulness really ceases whenever it is stressed above the elastic limit.

Factor of Safety is usually defined to be the ultimate or breaking stress divided by the working stress. Thus if the average tensile breaking stress is 30 tons per square inch and the allowable or working tensile stress is 9 tons per square inch then the factor of safety is about $3\frac{1}{3}$.

Although, theoretically, the working load on a tension member might be increased from three- to four-fold before it collapsed, the actual safety factor is much less than this ; because, when the load is increased to about 1.7 times its original value permanent distortion takes place, and the member ceases to have any further useful life. So arises the expression that a working stress of 9 tons per square inch represents a " factor of safety of 1.7 on the minimum yield stress " (with average values the factor is about 2).

Load Factor. It is thus apparent that two important load values occur when a structure is gradually overloaded to destruction :—

- (a) The first (and smaller) load which, when reached, causes the structure to have no further *useful* life.
- (b) The second (and larger) load which, when attained, causes complete *collapse*.

Hence the load factor is the number obtained by dividing that load

which if carried by the structure would cause it to have no further useful life by the load normally carried. This factor, or number, is not a ratio of stresses (in tons per sq. in.) but is a ratio of loads (in tons or pounds weight).

Load factors are greatly used in aeroplane design. With this particular type of structure accident-risk statistics may also have to be considered when fixing the appropriate load factor.

Strut formulæ give either the working or the ultimate end load in tons per sq. in. of end cross-sectional area, without mentioning the larger extreme fibre stress which may occur at, say, the mid-height of a long slender column where it bends away from the centre line. If the ultimate load is stated then the working load is taken as a definite fraction of the ultimate load. The ultimate load divided by the working load is sometimes erroneously termed "factor of safety," but, as this term is definitely reserved for stress ratio and not load ratio, the correct phrase is load factor. This point is more fully referred to in Vol. II., under the subject heading of Perry Formula.

Load factors and factors of safety have a common aim in that they express the margins of safety (or, conversely, the risks of failure) in structures; and both terms, in turn, depend upon the economy or resources of a nation, since unduly large load factors and factors of safety mean a gross misuse of material. Briefly, the present day use of the terms load factor and factor of safety depends upon the type of structure and the material used in its construction.

Working Stress is defined to be the force (in tons or pounds) on a member divided by the cross-sectional area in square inches. For a tension member it is the net area and for a compression member the gross area, which is considered. If a choice of materials is available then the strongest material will require the least cross-sectional area, *i.e.*, the stronger the material the higher is the working stress used in the calculations.

The following have to be considered when fixing the permissible, allowable or working stress, as it is variously termed.

(a) The variable quality of the material (*e.g.*, timber) and the standard of workmanship during fabrication and erection.

(b) Whether the load is gradually or suddenly applied.

(c) The duration and number of applications of the load at maximum intensity, *e.g.*, a wind load.

(d) Whether the structure is a temporary one or a permanent one.

(e) The degree of accuracy with which the forces in the structure can be ascertained.

The exact effect of every load cannot be definitely calculated,

nor can even the correct numerical values of many of the loads be foretold, *e.g.*, live load forces caused by locomotives, and wind loads. Again, stresses arise from the interaction of one part of a structure with another, which can be estimated only approximately. Assumptions are also made in the calculations of the primary stresses which are known to be not quite correct, *e.g.*, a joint with many wide-pitched rivets in it is assumed to be connected and held in place by one single and central pin.

(f) Corrosion and future maintenance.

(g) Various combinations of loads and the degree of probability of all conditions of maximum loading occurring simultaneously, *e.g.*, dead load plus live load plus impact plus wind load.

A definite margin of safety must be maintained to guard against all such uncertainties and hence the reason for the factor of safety or the load factor mentioned above.

As a more detailed example of the difficulties encountered in designing consider the case of a railway bridge. An electric train has a different effect upon a bridge from one where the power is supplied by a steam locomotive, in fact, the various types of ordinary steam locomotives have vastly different effects at different speeds upon the bridge stresses. The type of bridge floor used has an important bearing upon the stresses in the main girders, *e.g.*, the ballasted track is less severe than the tied floor. Next, there is an uncertainty as to the exact numerical value of all the effects now loosely but collectively grouped under the term "impact," and so it goes on.

The commonest method employed when designing a bridge is to calculate the forces due to the live loads as if they were momentarily static loads, and then to increase these forces by a variable allowance for "impact." In the case of a small-span bridge or a cross girder the load comes upon it practically instantaneously. On the other hand, a large span only receives its maximal main boom live load forces gradually as the train advances on to the bridge, and hence the impact factor is reduced as the span increases.*

Another method which used to be satisfactorily employed was to vary the working stress (and hence the factor of safety) according to the nature of the load. The tensile working stresses allotted to wind, dead and live loads could be, say, 12, 10 and 8 tons per sq. in. (of mild steel), respectively (*i.e.*, factors of safety of $2\frac{1}{2}$, 3 and $3\frac{3}{4}$ on the average ultimate tensile stress of 30 tons/sq. in.).

* For further information as to the effects of impact upon bridges, consult "*The Report of The Bridge Stress Committee*," issued by THE DEPARTMENT OF INDUSTRIAL RESEARCH. This publication should be referred to by everyone interested in the design of railway bridges.

A short discussion on impact is given in "*Influence Lines*," pages 173-181.

Thus if a workshop member carries a wind load of 6^T , a dead load of 20^T and a live load from cranes of 24^T the procedure employed is as shown on the accompanying table.

—	Actual Load, Tons.	F. of S.	Total Ultimate Load, Tons.
Wind . . .	6	$2\frac{1}{2}$	15
Dead . . .	20	3	60
Live . . .	24	$3\frac{3}{4}$	90
	—		—
	50		165
	—		—

Hence the final factor of safety to use is $165 \div 50 = 3.3$. If the member is under tension then $F_t = \text{ult.} \div \text{f. of s.} = 30 \div 3.3 = 9.09^T/\text{sq. in.}$, and the net area of metal required is $50 \div 9.09 = 5.5 \text{ sq. in. net.}$ The load factor for struts can be dealt with in a similar manner.

Still another method is to increase the actual live load in order to obtain an equivalent static load. A good example of this is the Ministry of Transport's Standard Load for Highway Bridges (June, 1922), where the actual wheel loads have been increased by 50 per cent., irrespective of speed, span of bridge, type of bridge floor, etc.

There are other methods and formulæ sometimes recommended for the derivation of the working stress, such as Launhardt-Weyrauch, the "Dynamic," etc., but these, however, are hardly ever used in modern design practice.

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WORKING STRESSES

Specifications :—See above.

CHAPTER X

BEAMS

$$\frac{M}{I} = \frac{f}{y} = \frac{E}{R} \cdot \cdot \cdot \cdot \cdot 1$$

Where M = Bending moment in inch tons (or foot tons).

I = Moment of inertia of beam's cross-section in inches⁴
(or feet⁴).

f = Stress in tons per square inch (or tons per square foot).

y = Distance from neutral axis to the fibre considered, in inches (or feet).

E = Young's modulus in tons per square inch (or tons per square foot).

R = Radius of curvature of the bent beam, in inches (or in feet).

Notwithstanding that this formula is perhaps the most important one employed in the design of structures, beginners often make absurd mistakes through neglecting to specify the units employed. This is not altogether their fault, because many text-books on mechanics prove the formula as a mathematical problem without clearly stating the units which are employed, although units are of paramount importance to the engineer. In steelwork calculations in this country stresses are always given in tons per square inch, and for reinforced concrete and timber work in pounds per square inch. If f is required in tons per square inch, then all the other items of equation 1 must be expressed in tons and inch units.

Thus $\frac{M}{I}$ is $\frac{M \text{ in inch tons}}{I \text{ in inches}^4} = \frac{M}{I} \left(\frac{\text{tons}}{\text{inches}^3} \right)$.

Similarly, $\frac{f}{y} = \frac{f \text{ in tons/sq. in.}}{y \text{ in inches}} = \frac{f}{y} \left(\frac{\text{tons}}{\text{inches}^3} \right)$,

since "per" or "/" is equivalent to the sign of division. The words ton and inch can be "cancelled out top and bottom" as with ordinary arithmetical figures. Again,

$$M = \frac{fI}{y} \text{ is in } \frac{(\text{tons/sq. in.}) \times \text{inches}^4}{\text{inches}} = \text{tons} \cdot \text{inches} \quad . \quad . \quad . \quad 2$$

When y is the distance to the extreme fibre from the neutral axis, the term $\frac{I}{y}$ is constant for that particular section. It can therefore be represented by one symbol Z , known as the section modulus.

$$\therefore \frac{M}{f} = \frac{I}{y} = Z \text{ in inches}^3 \quad \dots \dots \dots 3$$

In the cases of joists and channels the distance y from the xx neutral axis to the extreme fibres is the same, and, therefore, the fibre stress is numerically the same (but opposite in sign, i.e., tension

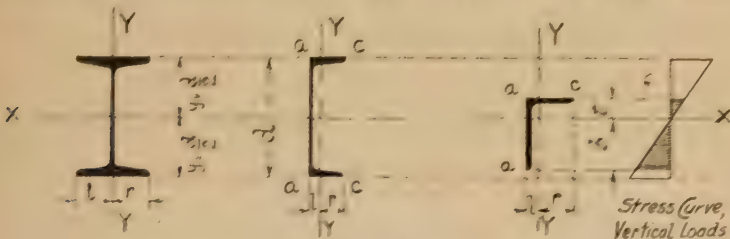


FIG. 154

and compression) on the bottom and top flanges, by equations 1 and 3.

The angle, however, has two values for y , viz. t and b . Hence, Z top or $Z_t = \frac{I}{y_t}$ and Z bottom $= Z_b = \frac{I}{y_b}$ from equation 3.

The fibre stress on the top fibre of the angle is $f =$ bending moment $\div$ section modulus $= M \div Z_t$, and on the undermost fibre it is $M \div Z_b$.

If the load, instead of acting vertically along the yy axis, now acts horizontally along the xx axis, the fibre stress is numerically the same at all four corners of the joist. With the channel and the angle there is Z_t and a Z_r , consequently the stress intensities at the points a are the same, but are of different value and of opposite sign to the stresses at points c . In the listed moduli of unsymmetrical sections it is always the smaller Z which is given.

Approximate Section Modulus of a Joist may be obtained by dividing the product of joist depth and weight by :-

- (a) 10 for joists less than 12" in depth and
- (b) 10.5 for joists 12" and over in depth. Examples :-
- (a) $10" \times 5" \times 30 \text{ lb.}$; $Z = 29.25$, against $10 \times 30 \div 10 = 30$.
- (b) $14" \times 6" \times 57 \text{ lb.}$; $Z = 76.19$, against $14 \times 57 \div 10.5 = 76$.

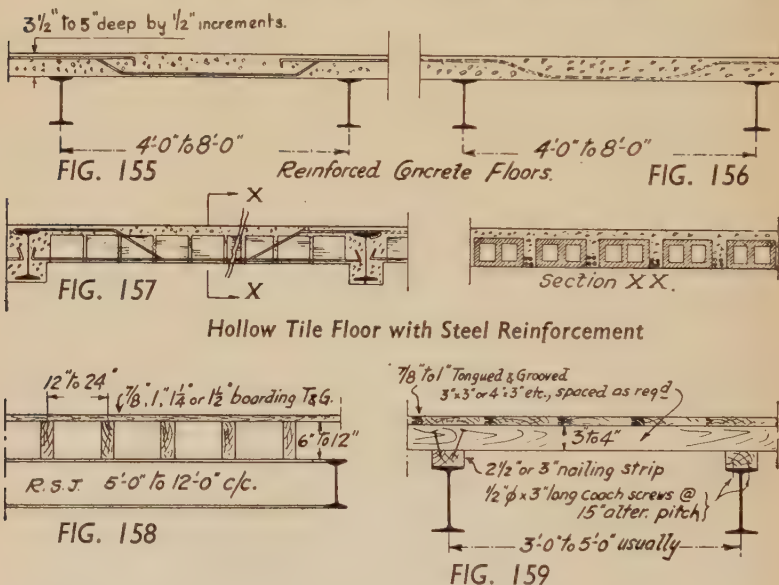
The rule is empirical and no units are involved ; it is only approximate but is useful when a section list is unobtainable.

Approximate Section Modulus of a Channel. With channels the constants are approximately 11 for channels of 11 in. and under in depth, and 12 for channels of 12 in. and over in depth. Examples:—

- (a) $9" \times 3\frac{1}{2}" \times 22.27$ lb.; $Z=18.36$, against $9 \times 22.27 \div 11 = 18.2$
 (b) $15" \times 4" \times 36.37$ lb.; $Z=46.55$, against $15 \times 36.37 \div 12 = 45.5$.

EXAMPLE OF SIMPLE BEAM CALCULATIONS: FLOORS

Design a floor to carry a load of 150 lb. per square foot in addition, of course, to its own dead weight; working stresses as on p. xiv.



The floor is of reinforced concrete (see R.C. trade catalogues for such flooring), 5 in. thick, including granolithic surface, and is carried on R.S.J.'s.

The mode of calculating and the general layout of the beams are applicable to all the types of flooring illustrated by Figs. 155 to 159. Where beams are encased in concrete the dead weight of such casing must be taken into account. Suspended or false ceilings should have their weight estimated at the rate of 8 to 12 lb. per square foot of ceiling area. (Reinforced concrete 150 lb., and ordinary concrete 140 lb. per cubic foot.)

In laying out the key plan of the floor beams previous to calculating it is economical to have :—

- (1) The major portion of the steelwork of small span for easy handling.
- (2) Equal spacing of beams.
- (3) If possible, no concentrated load at mid-span.

The concrete slab floor being cast *in situ* upon the steel beams exerts an adhesive and frictional resistance to any lateral movement of the upper or compressive flanges. Because of this continuous lateral support the working stress $F_c = F_t = 10^7$ per sq. in.

The beams are free-ended or simply supported, the end cleats being too shallow to afford any fixity. The maximum bending moment, therefore, occurs at mid-span and the maximum shear at the ends, both occurring simultaneously when the floor is completely loaded.

Light Beams or Stringers. Span 12 ft. 6 in. at 5 ft. c/c., Fig. 160.

Loading.

Superimposed load	= 150 lb. per sq. ft.		
5" R.C. floor	= (5		
÷ 12) × 150	= 63 lb. per sq. ft.	= lb./sq. ft.	213
External load on one stringer	= 213 lb./sq. ft. × 5' × 12.5'	= tons	5.94
Maximum <i>B.M.</i> from above	= $Wl \div 8 = 5.94^r \times 12.5' \times 12" \div 8$	= in. tons	111.4
111.4 ÷ <i>F</i> = approximate modulus, and so an idea of the weight of the beam is obtained from the section list, Table 5, Appendix.			
Weight of beam	= 18 lb./ft. × 12.5'	= ton	0.1
Maximum <i>B.M.</i> due to beam's own weight	= $0.1^r \times (12.5' \times 12") \div 8$	= in. tons	1.9
Total <i>B.M.</i>	= 111.4 + 1.9	= in. tons	113.3
Total Max. Shear	= reaction = $\frac{1}{2} (5.94^r + 0.1^r)$	= tons	3
Modulus req.	= $M \div F_t = 113.3 \div 10$	= in. ³	11.33
Web area req.	= shear ÷ $F_w = 3 \div 6.5$	= sq. in.	0.46

Section. Each of the following three rolled steel joists has the required section modulus but the last is adopted because it provides the necessary depth for the notch and cleats shown on Fig. 164 (a) :—

<i>viz.</i> , 1 R.S.J.	6" × 4½" × 20 lb.	<i>Z</i> = in. ³	11.57
and R.S.J.	7" × 4" × 16 lb.	<i>Z</i> = in. ³	11.29
but adopt R.S.J.	8" × 4" × 18 lb.		
(<i>M.</i> of <i>I.</i> = 55.6)		<i>Z</i> = in. ³	13.91

Web area, allowing for 2" notch at top

$$= (8'' - 2'') \times 0.28'' \text{ thick}$$

$$= \text{sq. in.} \quad 1.68$$

Deflection

$$\Delta = \frac{5 W l^3}{384 E I} = \left(\frac{5 \times 6 \times 150^3}{384 \times 13,000 \times 55.6} \right) \left(\frac{\text{tons} \times \text{in.}^3}{\text{tons per sq. in.} \times \text{in.}^4} \right) = \text{in.} \quad 0.37$$

Where W = total load = 6^T ; E = $13,000^T/\text{sq. in.}$; I = 55.6 in.^4
 The deflection should not exceed $\text{span} \div 325$. The calculated deflection of $0.37 \text{ in.} = \text{span} \div 406$ is therefore satisfactory. By the rule on p. 156 the deflection need not be calculated since F is less than 10 and depth $\div$ span is not shallower than $1 \div 19$.

The beam can be notched safely at the end because the top flange has no bending moment to carry at this point (Figs. 163 and 164).

Main Beams between Columns. Span 25 ft. @ 12 ft. 6 in. c/c. (Figs. 160 and 161.)

The loading on this beam consists of four pairs of concentrated loads (eight stringer reactions), in addition to a distributed load,

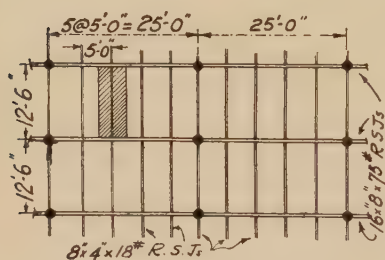


FIG. 160

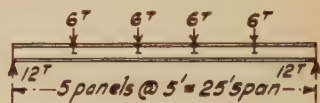


FIG. 161

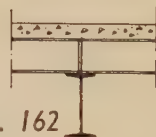


FIG. 162



FIG. 163

(1) from the beam itself, (2) from the concrete floor. The latter is small and its exact value cannot be definitely stated. Further, it can only be taken into account if the concentrated loads from the stringers be reduced proportionately. Since a concentrated load gives a larger bending moment than the same load uniformly distributed, it is safer, and at the same time easier, to neglect any local distributed load acting directly from the floor concrete on the main beam. The concentrated load from the floor stringers will be taken as if the stringer carried all the load on the shaded area of Fig. 160. This is equivalent to assuming that the stringers pass over the main beam (Fig. 162) instead of bosoming into it (Fig. 163). Further, the possibility of all the floor space being covered with merchandise weighing 150 lb./sq. ft. is remote, as access passageways must be maintained; the position of these spaces will vary from

time to time, so that it is but reasonable to assume all the floor loaded and thus considerably simplify the design.

Fig. 162 suggests that if the stringers were carried continuously over the main beam a lighter stringer would result, from the fact that the maximum bending moment in a beam continuous over several spans is less than that in the simply supported single span. This undoubtedly is true, but such an advantage can only be gained at the expense of head room, which is a more costly and therefore a more vital factor in the design.

The span of the beam is taken as being centre to centre of column shafts. This is on the safe side and permits the use of any type of column, wide or narrow.

Loading :—

The end reactions from each pair of stringers		
= 2 @ 3 ^r	= tons	6
Maximum <i>B.M.</i> due to above panel loading		
(Fig. 161) is (12 ^r × 10' — 6 ^r × 5')		
= 90 ft. tons	= in. tons	1080
(Approximately $Z = 1080 \div 10 = 108$;		
roughly the R.S.J. weighs 65 lb. per ft.		
Weight of beam self = 65 × 25 = 1625 lb.,		
neglecting cleats, see Table 5, Appendix	= ton	0.72
Maximum <i>B.M.</i> due to beam self = $WL \div 8 =$		
$0.72 \times 25 \times 12 \div 8$	= in. tons	27
Total Max. <i>B.M.</i> at mid-span = 1080 + 27	= „	1107
Modulus required = 1107 ÷ 10	= in. ³	110.7
Web area required = end shear ÷ F_w		
= $(12^r + \frac{1}{2} \text{ of } 0.72^r) \div 6.5 \text{ }^r/\text{sq. in.}$	= sq. in.	1.9
Modulus given. — One R.S.J., 20" × 6½" ×		
65 lb. (<i>M.</i> of <i>I.</i> = 1226)	$Z = \text{in.}^3$	122.6

Note.—There is practically no diminution in the value of the listed section modulus as the beams are holed in the web and not in the flanges.

Web area given, 20" × 0.45" thick	= sq. in.	9
-----------------------------------	-----------	---

Deflection. Since the joist depth is $\frac{1}{15}$ of the span, and the maximum fibre stress is slightly less than 10 tons/sq. in., the beam need not be investigated for deflection. See a later article in this chapter under the heading of Deflection of Beams, Ratio of Beam Depth to Span. A quick slide rule check, however, is obtained by assuming all the load uniformly distributed and then using the

$$\text{standard form of } \frac{5}{384} \frac{WL^3}{EI} = \frac{5 \times 24.72 \times 300^3}{384 \times 13,000 \times 1226} = 0.545''$$

The deflection obtained by this approximation is on the low side, but it is well under the permissible of $\text{span} \div 325 = 0.923''$, and so indicates that the beam is sufficiently rigid. The presumably true deflection is 0.659 in. as found by the more intricate calculation of Fig. 168. The actual deflection may be slightly less than the last figure owing to the monolithic nature of the floor.

Alternative Section is the R.S.J.

$16'' \times 8'' \times 75$ lbs. (M of $I = 973.9$) whose Z is $= \text{in.}^3$ 121.7,
and web area is $= 16'' \times 0.48''$ thick $= \text{sq. in.}$ 7.68

This joist is preferable to that found above, although it weighs 10 lb. more per foot run. It is 4 in. less in depth than the 20-in. R.S.J., which means a 4-in. saving on the height of the building. As the 10 lb. extra per foot run hardly affects the bending moment or end shear already found, the only point to check is the deflection, which, since the depth is now less, will be greater than that for the previous joist; *i.e.*

$$= 0.545'' \times \text{inverse ratio of the inertias} = 0.545'' \times \frac{1226 \div 973.9}{1} = 0.686''$$

Similarly, the true deflection $= 0.659'' \times 1226 \div 973.9 = 0.829''$
which is less than the permissible of 0.923 in.

Adopt $16'' \times 8'' \times 75$ lb. R.S.J. for the main beams.

Fastenings. Stringers to main beam. A rough guide to cleat thickness is that these should never be thinner than the web of the thinner joist to which they are riveted. They act as diminutive cantilevers and can be so calculated, but an experienced designer never does so. Where there is a cleat on each side of the joist web each may be of the same thickness as the web; if only one cleat, then it should be made slightly thicker than the joist web; and if the cleat has two rivet lines per leg, the cleat thickness should be still greater because of the longer lever arm to the centre of gravity of the rivet group on the outstanding leg.

Fig. 164. The 2-in. deep notch in the top flange of the light joist permits it to run into the web of the heavy joist without fouling (see Table 5, Appendix, for the clear depth of rolled joist webs). Similarly, anything riveted to the web of the light joist must stop short at a point 1 in. above the bottom flange so as to clear the bottom root fillet. The clear depth of the cleats is thus limited to 5 in., and this, in turn, limits the rivet diameter to $\frac{3}{4}$ in. or $\frac{13}{16}$ in. Thus, using the minimum end distance of $1\frac{1}{2}$ diameters and a 3 diameters pitch, $\frac{7}{8}$ in. diameter rivets would require a total clear depth of $1\frac{1}{2}$ diameters + 3 diameters + $1\frac{1}{2}$ diameters $= 6 \times \frac{7}{8}'' = 5\frac{1}{4}''$. The cleats will be riveted to the main beam previous to dispatch, and

open holes will be left in the web of the light joist for site rivets or bolts; the latter fastening is the more probable. This allows the main beams to be erected, and then afterwards the light beams are swung into position.

$\frac{3}{4}$ in. diameter rivet values:— $S.S. = 2.65^T$, $D.S. = 5.30^T$, bearing, $0.28'' = 2.52^T$ and $0.48'' = 4.32^T$; see values on p. xv.

Using a single cleat at B places the bolts thereat in $S.S.$ or 0.28 in. bearing; the joist web is thinner than the cleat.

Number of rivets required = $3^T \text{ load} \div 2.52^T = 1$.

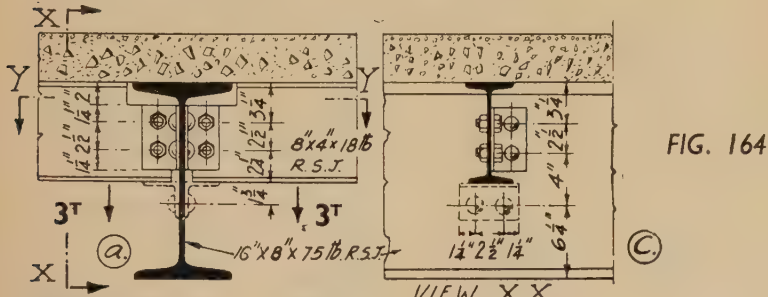
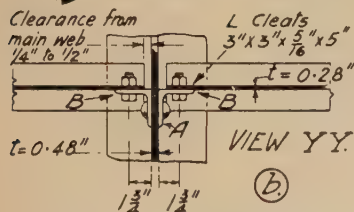


FIG. 164



VIEW X X.

VIEW Y Y.

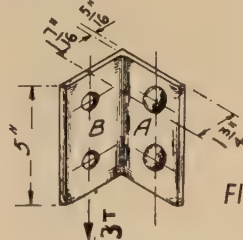


FIG. 165

RIVETS $\frac{3}{4}$ " dia.; (holes $\frac{3}{4}$ " dia.)
BOLTS $\frac{3}{4}$ " dia.; holes $\frac{13}{16}$ " dia.

If black bolts are used instead of rivets increase this number by 50 per cent. (see p. xv), i.e., say 2. In any case never use less than two fastenings.

As both bolts and rivets are taken at $\frac{3}{4}$ in. finished diameter, the holes for these must be $\frac{13}{16}$ in. and $\frac{3}{4}$ in. diameter respectively. Shop work would be facilitated if both sets of holes were made $\frac{13}{16}$ in. diameter, i.e., $\frac{3}{4}$ in. diameter bolts and $\frac{13}{16}$ in. diameter rivets.

The cleat adopted will be $3'' \times 3'' \times \frac{5}{16}'' \times 5''$, which has the rivet lines in each leg sufficiently far apart to prevent the fouling of the rivet heads at A with the bolts at B , as the depth of 5 in. does not permit the reeling of the A fastenings with the B fastenings. The load on the A fastenings is twice $3^T = 6^T$, and these are in $D.S.$ or 0.48 in. bearing. Giving two rivets at A provides for a direct load of $2 \times 4.32^T = 8.64^T$, which is much in excess of the actual direct load of 6^T .

Fig. 165. Calculating the cleats as cantilevers rigidly held to the main joist and carrying a concentrated load at the bolt lines *B* of 3^{π} , shows a bending moment at face *A* of $3^{\pi} \times 1\frac{7}{16}'' = 4.31$ in. tons. The resisting modulus at face *A* is that of a flat 5 in. deep $\times \frac{5}{16}$ in. thick $= \frac{1}{6} \times \frac{5}{16} \times 5^2 = 1.30$ in.³ The maximum fibre stress at the top and bottom of the cleat where the outstanding leg meets face *A* is $M \div Z = 4.31 \div 1.3 = \pm 3.31$ tons/sq. in.

Landing, or shelf, cleats are useful in supporting the light joists during erection. Sometimes their use becomes imperative when a

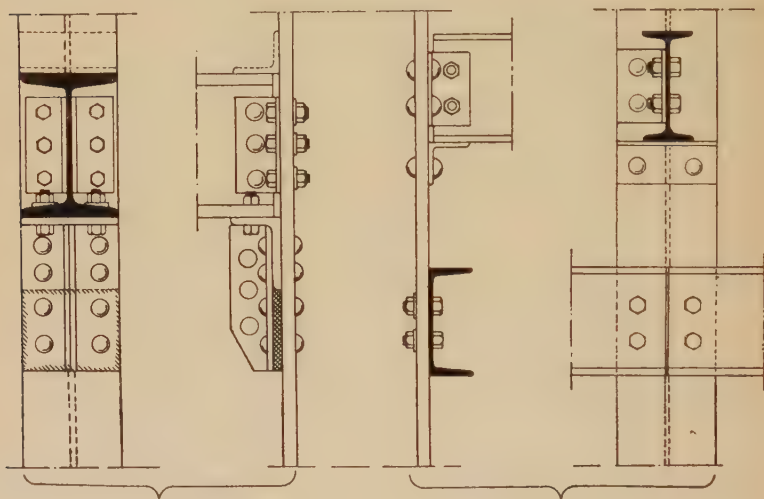


FIG. 166

FIG. 167

sufficient number of bolts to carry the end shear cannot be placed in the vertical cleats, and the end shear has to be countered by both the shelf and vertical cleat fastenings; in the case under discussion such a necessity does not arise. A $2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$ or $3'' \times 3'' \times \frac{1}{4}''$ angle, 5 in. long, with two rivets (indicated in broken line in Fig. 164), will easily support the dead load of the stringer until bolting up is completed. The bolting up of the bottom flanges of the light joists to these shelf cleats is a matter of opinion, but if carried out braces the whole structure laterally during erection. Once the concrete floor is laid the necessity for such bolts hardly exists. One or two bolts may be used in conjunction with bevelled washers on the sloping face of the bottom flange of the light joist.

If four vertical cleats instead of two had been used at the joint (*i.e.*, one cleat on each face of the thin webs) no gain in rivet values

at B would have been obtained, since the light joist web is so thin that the bearing value is actually less than the single shear value.

Main Beams to Columns. The number of bolts required to transfer the 12.36 tons end reaction (12^T of Fig. 161 plus half the beam's own weight of 0.72^T) of the main beam into the column shaft is found in a manner similar to that used for the stringers to main beams. A few typical beam to column details are illustrated in Figs. 166 and 167.

Flange Width. The upper or compression flange tends to buckle sideways because of strut action. Obviously a narrow compression flange with few lateral supports is more prone to buckle than a broad flange with the same supports. In the present instance all the upper flanges are well supported laterally by the frictional and adhesive contact with the concrete floor. Taking adhesion at 100 lb. per sq. in. this supporting force per foot run of the $16'' \times 8'' \times 75$ lb. R.S.J. = $8'' \times 12'' \times 100$ lb./sq. in. = 4.3^T . If the upper flange of this joist had no lateral support whatsoever throughout its span l of 300 in. the working compressive stress F_c would be reduced (F_t remains constant at 10^T /sq. in.). From Table 5, Appendix, $l/k_y = 300/1.76 = 170$. Also $k_x/k_y = 6.64/1.76 = 3.77$. From p. xiv, $K_1 = 1.31$ and hence $F_c = 1000K_1 \div l/k_y = (1000 \times 1.31) \div 170 = 7.7^T$ /sq. in. Required modulus = $M \div 7.7$ (and not $M \div F_t$ of 10) = $1107 \div 7.7 = 144$. (R.S.J. $18'' \times 8'' \times 80$ lb., $Z = 144$ is not suitable as F_c works out at 6.8^T /sq. in.)

DEFLECTION OF BEAMS

When the loading of a beam or girder is irregular, and of a form not covered by the standard formulæ, probably the easiest way to evaluate the deflection is as follows.

Load the beam or girder with the actual bending moment curve and find the resulting or second bending moment diagram due to this loading. This secondary bending moment curve is the deflection diagram for the beam, and an ordinate at any point represents, to scale, the vertical deflection of the girder at that point. This is known as the method of elastic weights.

In Fig. 168 the weight of the beam of 0.72^T is divided into four equal concentrations of 0.18^T instead of being uniformly distributed. This 0.18^T added to the 6^T from Fig. 161 gives the total panel load of 6.18^T . This causes but a very slight difference in the value of the maximum bending moment, as witness 1112 against 1107 previously used. The area of portion $A_1 = \frac{1}{2} \times 60'' \times 742$ in. tons = 22,260 in.² tons, and the centre of gravity of the area is 110 in.

from the centre line; similarly with the two remaining areas. The reactions R_L and R_R are now each equal to $A_1 + A_2 + A_3 = 111,240$ in.² tons. From the symmetry of the figure the maximum *B.M.* obviously occurs at the centre line and its value is $150'' \times R_L -$

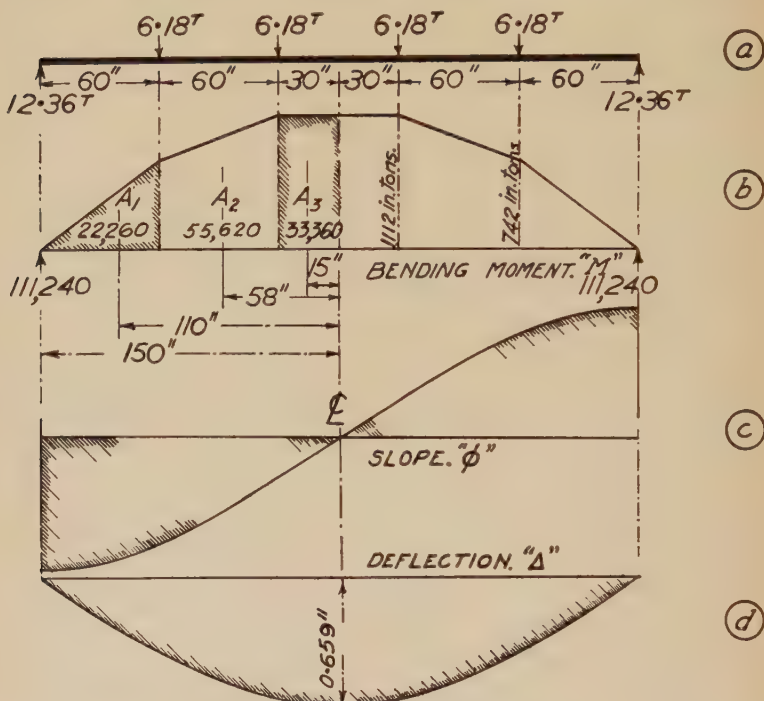


FIG. 168

$$110A_1 - 58A_2 - 15A_3 = 150 \times 111,240 - 110 \times 22,260 - 58 \times 55,620 - 15 \times 33,360 = 10,511,040 \text{ in.}^3 \text{ tons.}$$

Then this secondary *B.M.* $\div E.I.$ = deflection,

$$\Delta = \frac{10,511,040 \text{ in.}^3 \text{ tons.}}{13,000 \text{ tons per square inch} \times 1,226 \text{ in.}^4} = 0.659 \text{ in.}$$

Remember that "per" is equivalent to the sign $\div$ and that the units "cancel out" as with ordinary numbers.

In the present instance neither the slope curve, *i.e.*, the secondary curve of shear obtained from the primary *B.M.* as loading, nor the deflection curve requires to be drawn since, from symmetry, the maximum moment occurs at the centre line. It will be recalled that when the primary shear curve cuts the base line, *i.e.*, zero value, the

primary bending moment curve registers a maximum value. Similarly with the secondary shear and bending moment curves, for where the slope curve cuts its base line the deflection is a maximum, as will be seen from the figure. This property is of value when the loading is asymmetrical, because if the secondary shear or slope curve is sketched out the position of maximum deflection is indicated and the secondary bending moment or deflection calculated for this one point only. It is to be emphasized that the point of maximum deflection does not necessarily occur under a concentrated load. Thus if a single concentrated load is applied at a quarter point of the span of a beam the maximum deflection occurs neither at the centre line nor at the load point, but at a point distant $0.06 \times$ span from the centre line and on the same side of the centre line as the load.

A very simple, but extremely accurate, graphical construction for deriving the slope curve Fig. 168 (c) and the deflection curve Fig. 168 (d) is given in Chapter XVI. of *Influence Lines*.

Ratio of Beam Depth to Span is determined, apart from æsthetics, by the permissible deflection, which should not be greater than $\text{span} \div 325$ (*B.S. 449.—Building*). Usually the ratio is about $1/15$ for depth $\div$ span for heavy beam work, $1/20$ for medium and $1/25$ for light work. The following rule is, however, more satisfactory.

Let W = total load in tons ; l = span in inches ; E = Young's modulus of $13,000 \times$ sq. in.

I = the moment of inertia in in.^4 ; F = stress in tons/sq. in.

y = the distance in inches to fibre from the neutral axis. If to the outermost fibre then,

= $\frac{1}{2}$ beam depth = $\frac{1}{2} D$.

M = bending moment in inch tons = $Wl \div 8$ and $Wl \div 4$ in the following examples :—

$$\text{Since } \frac{M}{I} = \frac{F}{y} \therefore M = \frac{IF}{y} = \frac{2IF}{D} \quad . \quad . \quad . \quad 1$$

For a uniformly distributed load

$$\Delta = \frac{5Wl^3}{384EI} = \frac{Wl}{8} \left(\frac{5l^2}{48EI} \right) \quad . \quad . \quad . \quad 2$$

$$\text{Now } \frac{Wl}{8} = M = \frac{2IF}{D} \text{ by 1 } \therefore \Delta = \frac{2IF}{D} \left(\frac{5l^2}{48EI} \right) = \frac{Fl^2}{4.8ED} \quad . \quad 3$$

If F be limited to $10 \times$ sq. in., and Δ to $(\text{span} \div 325)$, equation 3 can be written

$$\frac{l}{325} = \frac{Fl^2}{4.8ED} \text{ or } D = \frac{677l}{E} \quad . \quad . \quad . \quad 4$$

Whence depth $D = 677l \div 13,000 = \text{span} \div 19 \quad . \quad . \quad . \quad 5$

Similarly for a concentrated load W at mid-span,

$$\Delta = \frac{Wl^3}{48EI} = \frac{Wl}{4} \left(\frac{l^2}{12EI} \right)$$

and depth $D = 542l \div 13,000 = \text{span} \div 24 \quad . \quad . \quad . \quad . \quad 6$

It is immaterial how a total load of W tons be applied to a beam (as one or as a series of concentrations, as distributed, partially distributed, or as a mixture of distributed and concentrated loads) the deflection must lie between the two values given above.

Rule. Therefore, it can be stated that if the extreme fibre stress does not exceed $10^7/\text{sq. in.}$, and if the beam depth is not shallower than $\text{span} \div 19$, then the deflection need not be calculated, since it will not exceed $\text{span} \div 325$.

LINTEL BEAMS

The following example should present no difficulty if the design of the floor beams has been followed. Find a suitable rolled steel section to carry a masonry wall 12 in. thick by 15 ft. high over a 15 ft. clear opening (masonry at 150 lb. per cubic foot).

Permissible stresses.	See p. xiv.		
	Bearing pressure on sand-stone end bearing blocks	= tons/sq. ft.	20
Effective span.	Allow a 1 ft. length of bearing at each end. Span c/c of bearings	= ft.	16
Weight of wall	$= 16' \times 15' \times 1' \times 150 \text{ lb./ft.}^3$		
	$\div 2240 \text{ lb.}$	= tons	16
Weight of beam	is estimated at	,,	0.5
Total weight	uniformly distributed	= ,	16.5
Maximum <i>B.M.</i>	$= Wl \div 8 = 16.5 \times (16 \times 12) \div 8$	= in. tons	396
<i>Z</i> required	for tension $= 396 \div 10$	= in. ³	39.6
Shear area required	$= \text{end shear} \div F_w = (\frac{1}{2} \text{ of } 16.5 \times 1)$		
	$\div 6.5$	= sq. in.	1.27
Adopt a 10" $\times$ 8" $\times$ 55 lb. R.S.J.	Broad flange to give good seating		
	($I = 288.7$), <i>Z</i>	= in. ³	57.74

With this joist the laterally-unsupported span must exceed 23 ft. before F_c is reduced below

10^7 /sq. in. by formula, p. xiv, *i.e.*, F_c = tons/sq. in. 10

Shear area = beam depth $\times$ web thickness
 given = $10'' \times 0.4''$ = sq. in. 4
 Δ permissible = span $\div 325 = 16 \times 12 \div 325$ = in. 0.59

Δ actual = $\frac{5 \times W \times l^3}{384 \times E \times I}$
 = $\frac{5 \times 16.5 \times (16 \times 12)^3}{384 \times 13,000 \times 288.7}$ = " 0.40

Bearing at ends. Given 1 ft. long $\times$ 8 in.
 flange width on bed block = sq. ft. 0.66

Required, for sandstone,
 = $(\frac{1}{2} \text{ of } 16.5^7) \div 20^7$ /sq. ft. = " 0.41

Alternative Section. Two R.S.J's. side by side, 12"
 $\times 5'' \times 32$ lb.; $Z = 2 @ 36.8$ = in.³ 73.6
 The M of $I = 2 @ 221.1$ = in.⁴ 442.2

The I to use in the formula for F_c , p. xiv, is the very large one about the vertical line midway between the joists of Fig. 170.

The two upper flanges are tied to each other by means of the masonry, which serves as a continuous tie plate fastened to the



FIG. 169



FIG. 170

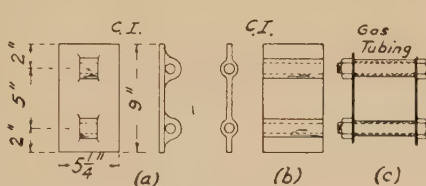


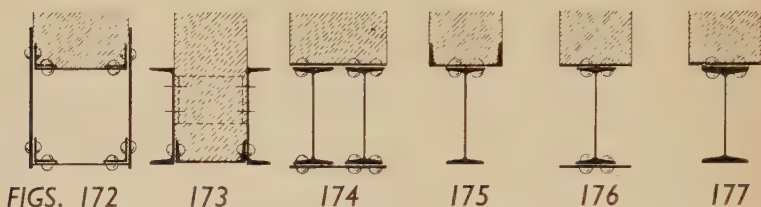
FIG. 171

steel by the adhesion of the mortar. In addition, cast iron or mild steel (pressed) separators will be used at 4 ft. centres. The usual spacing runs from four to six times the beam depth. A separator will be placed at each bearing, so that there will be a total of five separators in the lintel beam. For sizes and weights of these separators see trade catalogues.

The separators keep the beams together laterally, but little faith can be placed upon their acting as diaphragms equalizing the load between the joists. Each beam, due to the clearance of the bolts in the holes of the casting, is free to deflect without obtaining much help from its neighbour, and should one beam deflect more than the

other it will result in a cracked wall. Because of this some designers economize in weight by using bolts placed inside gas tubing for separators; they are just about as effective and cost less than the castings, which are illustrated in Fig. 171.

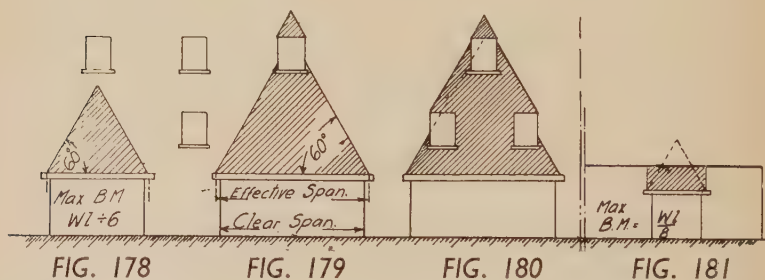
Other types of built-up lintel girders are shown in Figs. 172 to 177. Generally speaking, the first three are for large spans. When the section is unsymmetrical the centre of gravity of the section must



TYPES OF LINTEL GIRDERS

be found, and then the Z top and Z bottom. Fig. 175 is the cross-section of a simple and very efficient form of lintel girder, the Z's of which are easily obtainable. The pitch of the rivets in the built-up sections are found from the formula $q = SG \div Ib$, a numerical example of which is given further on in the present chapter.

Diaphragms, consisting of short vertical lengths of the same



channel section as is employed for the bottom of the lintel, are usually riveted to the webs of the vertical channels of Fig. 173. An alternative is $6" \times \frac{3}{8}"$ tie plates fastened by two rivets to each of the upper flanges of the vertical channels. The spacing of the diaphragms is the same as for the separators previously mentioned.

When lintel girders carry a wall unbroken by windows there is an arch action within the brick or masonry work which relieves the girder of some of the load. The assumption usually made is that

the girder carries only the hatched equilateral triangle of wall (Fig. 178). The remaining figures (179 to 181) are special cases of this rule; the shaded areas illustrate the active loads on the lintel girders. When building over a steel lintel it is better not to raise the brickwork too rapidly, for with a green wall there is the possibility of more than the equilateral triangle of loading acting on the beam.

The maximum bending moment with triangular loading is $Wl \div 6$, where W is the total load. The foregoing example took a liberal view of the possible load and is therefore on the safe side; the bending moment with triangular loading is 236 in. tons + 12 in. tons due to dead weight of lintel itself ($Wl \div 8$).

DECREASING AND INCREASING THE STRENGTH OF A BEAM

Beams with Holes. The listed moduli and moments of inertia are for gross sections without holes. Fig. 182 shows a joist with holes in the upper or compression side of the neutral axis, then :—



FIG. 182

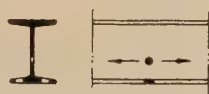


FIG. 183

(1) If these are rivet-filled no deduction need be made from the gross Z and M . of I . of the beam, as the rivets, on the assumption that they completely fill the holes, transfer the thrust across from one side of the gap to the other. (2) If bolt-filled (with a $\frac{1}{16}$ in. clearance in the diameters) the holes are not plugged, and deduction must be made from the Z and M . of I . The holes in the joist of Fig. 183 are on the lower or tension side of the N.A. In this case, as indicated by the arrows, the opposite sides of the holes tend to separate, and no matter whether the holes be rivet- or bolt-filled, deduction must be made from the gross Z and M . of I . of the beam.

In the cross-sections of both of these figures the neutral axis and centre of gravity line is no longer half-way up the section, but is nearer the unholed flange, where the heavier mass of metal lies; while at a section past the holes the N.A. and C. of G. line returns to the mid-depth position. It is common practice to take the neutral axis as always being at mid-depth, where it really lies for the major length of the beam, so eliminating the irksome calculations

necessary in finding the new centre of gravity and moment of inertia of the beam.

When beams are holed on the lower side only, the foregoing practice is equivalent to deducting a similar set of holes from the upper portion of the beam, although none may exist there. When the holes are in the upper portion of the beam, and none out of the lower portion, then the gross Z and gross $M.$ of $I.$ may be used provided the holes are rivet-filled; if bolt-filled a similar set of holes should be assumed out of the lower portion of the beam, see method (c) under.

To appreciate this statement better, consider the ideal beam of

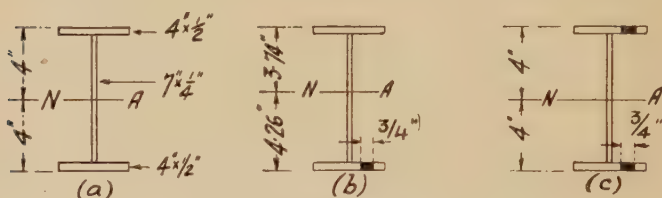


FIG. 184

Fig. 184, which has been used in place of an R.S.J. because of the facility with which the moment of inertia can be found.

- (a) Gross Section. $M.$ of $I. = 63.47 \text{ in.}^4$ Z top and bottom $= \text{in.}^3 15.87$
 (b) Net Section. $M.$ of $I. = 57.76 \text{ in.}^4$ Z top $= \text{,, } 15.45$
 and Z bottom $= \text{,, } 13.56$
 (c) Imaginary hole in top. $M.$ of $I. = 63.47 - 2 \text{ holes}$
 $(\frac{3}{4} \times \frac{1}{2}) \times (3\frac{3}{4})^2 = 52.92 \text{ in.}^4$ and Z top and bottom $= \text{,, } 13.23$

(The moments of inertia of the holes about their own axes are negligible.)

So that this method comes very close to the minimum Z found by the correct method of (b).

- (d) Another current method is to deduct from the gross Z of 15.87 that of one hole (Fig. b), the value of which is area of hole $\times$ distance from the N.A. Thus, $15.87 - (\frac{3}{4} \times \frac{1}{2}) \times 3\frac{3}{4} = \text{,, } 14.46$ which figure is practically the average of the two Z 's of (b).

- (e) Still another approximate method, which, however, is more applicable to deep girders than to R.S.J's.,

is to take the stress, found by using the gross Z , and multiply it by the ratio of gross flange area

to net flange area, i.e., $\frac{BM}{Z} \times \frac{(4 \times \frac{1}{2})}{(3\frac{1}{4} \times \frac{1}{2})}$. This

gives an equivalent modulus of Z gross $\times$
 $(3\frac{1}{4} \div 4) = 15.87 \times 3\frac{1}{4} \div 4 = \text{in.}^3 12.9$

Beam with Flange Plates. Usually it is the bending moment or the deflection which settles the size of a joist or channel, and only occasionally the shear; the last occurs when the beam is of short span and is heavily loaded, or where the beam is purposely kept shallow to give head room, as in warehouse design.

In general, therefore, the most economical beam is that wherein most of the weight is concentrated in the flanges, which, without undue thinning of the web, should be as far apart as possible. Hence, it follows that the capacity of a beam to withstand bending moment can be greatly increased by the addition of flange plates.

Example. To investigate whether the still shallower section of Fig. 185 could be used for the main beam of the floor already designed.

Gross $M.$ of $I.$ of $10'' \times 8'' \times 55$ lb. R.S.J. $= \text{in.}^4 288.7$

2 pls. $10'' \times \frac{3}{4}''$. $M.$ of $I.$ own axis (generally neglected)

$$= 2 \times \frac{1}{12} b d^3 = 2 \times \frac{1}{12} \times 10 \times 0.75^3 = \text{,, } 0.7$$

$$+ \text{area} \times \text{distance}^2 = 2 (10 \times 0.75) (5\frac{3}{8})^2 = \text{,, } 433.4$$

Total gross $M.$ of $I.$ $= \text{,, } 722.8$

— inertia of 2 holes $= 2 \times \text{area} \times \text{dist.}^2$

$$\frac{3}{4}'' \text{ diameters} = 2(1.53 \times 0.75) \times 5^2 \text{ approx.} = \text{,, } 57.4$$

Total net $M.$ of $I.$ $= \text{,, } 665.4$

Net section modulus $Z = \text{net } I \div y = 665.4 \div 5.75 = \text{in.}^3 115.7$

Web area given $= 10 \times 0.4 = \text{sq. in. } 4$

Against the requirements of 110.7 in.^3 and 1.9 sq. in. respectively.

Obviously, however, the deflection rules this section out because the $M.$ of $I.$ is 723 against 973.9 of the adopted 16-in. R.S.J., and so the deflection will be considerably greater. With the plated beam the deflection $= 0.829''$ of the $16''$ joist $\times 973.9 \div 723 = \text{in. } 1.12$. The permissible deflection was $25' \times 12 \div 325 = \text{in. } 0.923$.

Deflection and Gross Areas. The deflection is always calculated upon the gross moment of inertia, because when a structural element elongates, all the metal composing that element must elongate. Thus, consider a single flat bar suspender 4 in. wide and

$\frac{1}{2}$ in. thick with a 1 in. diameter hole in it, then when it stretches all the metal whose gross area of cross-section is 2 sq. in. must lengthen and not only the metal on either side of the hole, the area of which metal is $1\frac{1}{2}$ sq. in. net.

Curtailment of Beam Flange Plates. For the purpose of this paragraph assume the plated joist section to be satisfactory. When dealing with dead load it is common practice to assume that the dead load of a beam or plate web girder is concentrated at the panel points, instead of being uniformly distributed. The weight of the $10" \times 8"$ joist + two plates $10" \times \frac{3}{4}"$ + rivet heads = 55 + 51 + say 3 = 109 lb. per foot run (total weight, 1.2^T), giving a

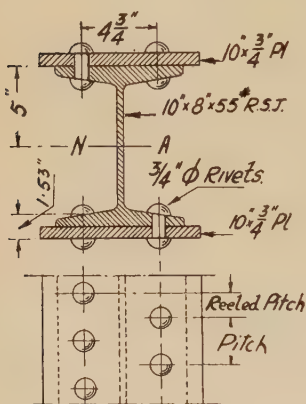


FIG. 185

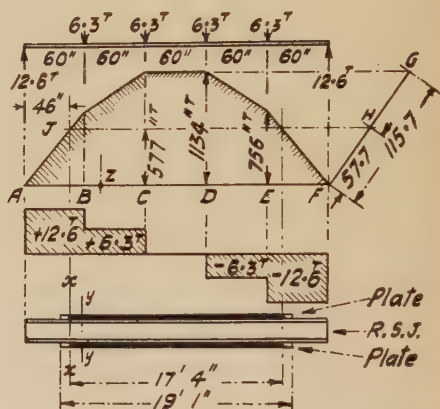


FIG. 186

panel load of 0.3 ton ; while the panel load due to merchandise and concrete floor was 6^T , Fig. 161.

Hence the total panel load = $6 + 0.3$ = tons 6.3

The bending moment at A and F is zero, Fig. 186 ;

at B and E it is $12.6^T \times 60''$ = in. tons 756

and between C and D it is constant at $12.6^T \times 120'' - 6.3 \times 60''$ = , , 113.4

The rate of increase of moment between A and B

is $756 \div 60''$, i.e., per inch run of beam = , , 12.6

The Z of $10"$ joist without plates

= in.³ 57.7

and the $B.M.$ it can carry = $ZF_t = 57.7 \times 10$ = in. tons 577

This occurs at a distance from each end of beam

= $577 \div 12.6$ = in. 46

Between either of these points and the adjacent column the beam

is strong enough to carry the bending moment unaided ; the flange plates need extend, therefore, only between these two points, *i.e.*, a length of 25 ft.—2 ends at 46 in. = 17 ft. 4 in. Specifications demand that the flange plates be extended beyond the theoretical end for such a length as will contain either (a) three rivets in each rivet line or (b) a sufficient number of rivets to develop half the plate strength.

The reasons underlying these rules are :—

(1) At the point of cut-off there is an instantaneous change of section modulus, but it does not follow that the distribution of stress will keep pace with the sudden change of section ; *e.g.*, at *xx* (Fig. 186) the bending moment is 577, joist *Z* 57·7 and flange extreme fibre stress 10 tons per square inch. At *yy*, say 2 in. to the right, the bending moment is still approximately 577, the *Z* is 115·7, while the extreme fibre stress on the plates = $577 \div 115·7 = 4·99$ tons per square inch. If the latter stress exists, then the stress on the extreme fibres of the joist is proportional to their distance from the N.A., *i.e.*, = $4·99 (5'' \div 5·75'') = 4·34$ tons per square inch. It is not conceivable that in such a short distance as *xy* the stress in the flange of the joist should drop from 10 down to 4·34 tons per square inch, and then gradually rise again to its maximum value at mid-span of 10 ($5'' \div 5·75'' = 8·69$ tons per square inch. The change must be gradual, and must take place through the medium of the riveting. The additional length of plate permits of this diffusion of stress to take place before the point is reached where the plates are really required.

(2) Another reason for the continuation of the flange plates past the point of theoretical curtailment is provided by moving wheel loads. The wheel bases for concentrated loads, railway or road, vary, and with them their individual bending moment parabolas. The point of theoretical cut-off is probably fixed by one type of loading (assumed or actual), whereas another type, but little different, will have its bending moment parabola slightly to one side or other of the point of cut-off. The advantage of the additional length of plate is now apparent.

Number of End Rivets.

Plate net area = $(10'' \times \frac{3}{4}'' - 1 \text{ hole } \frac{3}{4}'' \times \frac{3}{4}'')$	= sq. in.	6·9
At 10 tons/sq. in. this represents $6·9 \times 10$	= tons	69
One $\frac{3}{4}$ in. diameter rivet in <i>S.S.</i> , see page xv.	= „	2·65
No. of rivets to develop half plate strength =		
$\frac{1}{2} (69 \div 2·65)$	= rivets	13
More recent specification :—3 rivets/rivet line	= „	6

Additional plate length = 6 reeled pitches at 1½" plus 1½" end distance, <i>i.e.</i> (about)	= in.	10.5
Overall length of flange plate is 17' 4" plus two ends @ 10½", <i>i.e.</i> (about)	=	19' 1"

Allowing for the width of the columns the plates *might* be carried the full length of the joist in this particular case.

Table 11, Appendix, gives a reeled pitch of 4.71 in. as being the minimum to adopt to prevent diagonal tearing, which, however, need not be investigated, there being a large excess of metal (as witness a flange plate stress of 4.99 tons/sq. in.) at this particular position. The rivet pitch should be kept small at the ends of the plate in order to develop the half strength of the plate in as small a distance as possible, and then increased as the centre line (position of minimum shear) of the span is approached ; see succeeding article on Rivet Pitch.

The calculation for finding the theoretical point of cut-off was considerably simplified by the fact that the bending moment diagram consists of straight lines. The following graphical method is much simpler and is extremely useful when the bending moment diagram is composed of curves. With point *F* as centre, swing a decimal scale round until *FG* registers 11.57, representing the net section modulus of 115.7. This may be on the eighths scale, or ¼ in. scale, or even in centimetres, whichever scale works into the distances *FG*, whose length is somewhat longer than the height of the bending moment diagram. This diagram, in turn, may be to any scale. In the original drawing, before reduction, the vertical or *B.M.* scale could be ⅛" = 100 in. tons ; linear scale (*i.e.*, length of beam *AF*) about ⅛" = 1' 0" ; while *FG* could be swung out on the decimal ⅛ in. scale. These scales are rather small for accurate work, but they suffice for purposes of illustration.

Along *FG* now prick off *H*, making *FH* = 5.77 on the scale of *FG*, representing the joist net *Z* of 57.7 in.³ Through *H* draw a parallel *HJ* to the base line *AF*. Then so long as the *B.M.* diagram lies between the parallels *AF* and *JH* the joist modulus is sufficient ; elsewhere the built-up section must be used. The point *J* is 577 in. ton units above *AF* as already found by calculation.

The Rivet Pitch of Plated Beams. The formula used is $q = \frac{SG}{16}$, and is proved in Chapter I., Vol. II., where :—

q = the horizontal shear intensity in tons per square inch.
 S = the total vertical shear, at the section considered, in tons.

I = the moment of inertia of the total gross section in inches.⁴

b = the gross breadth of the horizontal layer at the part considered.

G = the first moment (*i.e.*, gross area $\times$ distance) about the N.A. of all the area of metal above the horizontal layer considered.

Another common form for this formula is $q = Say \div Ib$, where $ay = G$.

Example. To find the necessary pitching of the rivets for the foregoing plated beam. According to Fig. 186 the vertical shear S is constant at 12.6 tons between A and B . I = the gross $M.$ of $I.$ of 723 in.⁴ b = either the 8 in. width of the beam or the 10 in. width of the flange plates; it is immaterial which b is used, because in a 1 ft. length of the section the total force shearing the plates horizontally along the flanges of the joist is $q \times$ area per foot run =

$$q \times b \times 12 = \frac{SG}{Ib} \times b \times 12 = \frac{12SG}{I} \text{ the term } b \text{ cancelling out.}$$

$$G = \text{area} \times \text{distance} = (10'' \times \frac{3}{4}'') \times 5\frac{3}{8}'' = \text{in.}^3 \quad 40.3$$

$$\text{Hence shear per horizontal foot} = 12SG \div I$$

$$= 12 \text{ in.} \times 12.6^T \times 40.3 \text{ in.}^3 \div 723 \text{ in.}^4 = \text{tons} \quad 8.43$$

$$\text{Value of } \frac{3}{4}'' \text{ dia. rivet } (\frac{3}{4}'' B = 6.76^T) \text{ is } S.S. = \text{,,} \quad 2.65$$

$$\text{No. of rivets per ft.} = 8.43^T \div 2.65^T = \text{,,} \quad 3.2$$

Portion AB. Required reeled pitch in a 12 in.

$$\text{length of either flange} = 12 \div 3.2 = \text{in.} \quad 3.75$$

Any reeled pitch not exceeding this withstands

$$\text{the shear. Reeled pitch adopted} = \text{in.} \quad 1.5$$

Obviously $\frac{7}{8}$ in. dia. rivets would require to be wider pitched and are, therefore, not used.

$$\text{Portion BC. All floor loaded, shear} = \text{tons} \quad 6.3$$

This is not the maximum shear in the panel, which only occurs when portion FZ alone is loaded with live load, as explained under. The max. shear

$$= \text{,,} \quad 6.83$$

$$\text{Shear per horizontal foot} = 12SG \div I =$$

$$12 \times 6.83 \times 40.3 \div 723 = \text{,,} \quad 4.57$$

$$\text{The reeled pitch required now} = \text{in.} \quad 7$$

$$\text{Reeled pitch adopted} = \text{,,} \quad 2$$

Portion CD. The shear in this panel is zero when the floor is fully loaded. Maximum shear (2.1^T) occurs when the live load covers half the span from either column up to the centre line. As the rivet

pitch will be still wider than that of BC the shear will not be investigated further. (Reeled pitch adopted, 3 in.)

Actual Pitches Adopted. The maximum rivet pitch (*i.e.*, twice the reeled pitch) must not exceed $16t$ for parts in tension with a maximum of 8 in. and $16t$ for parts in compression with a maximum of 6 in. (*B.S. 449.—Buildings*). To simplify fabrication the pitches in the top and bottom flanges of any one panel are kept the same. The maximum pitch is thus limited to 6 in. although 16 times the $\frac{3}{4}$ in. thick flange plate gives 12 in. Adopt for the end 2 ft. length of plate, *i.e.*, up to point B , a 3 in. pitch; for the 5 ft. panel between B and C , a 4 in. pitch; and for the mid-panel CD a pitch of 6 in. The respective reeled pitches are, therefore, $1\frac{1}{2}$ in., 2 in. and 3 in., all well under the pitches demanded by calculation; while the pitch of 6 in. in the mid panel permits the net calculated Z to remain as calculated, there being no possibility of diagonal tearing; see Table 11, Appendix.

Maximum Shear in Panel BC, Fig. 186.—The point Z has its position fixed such that $BZ : ZC :: AZ : ZF$ or $\frac{BZ}{5 - BZ} = \frac{5 + BZ}{20 - BZ}$,

hence $BZ = 1\frac{1}{4}$ ft. and $ZC = 3\frac{3}{4}$ ft. This method applies to any panel. A simple proof of this rule is given in Chapter V. of "*Influence Lines*." In words it is:—Let Z divide the panel in the same ratio as it divides the span. In the case of the centre panel, Z' is at the centre line. Maximum positive shear is obtained by loading the right-hand portion FZ only, and maximum negative shear by loading the left portion A to Z .

Live load on beam per foot run @ 150 lb./sq. ft.

$$= 150 \times 12.5 \text{ lb. (Fig. 160)} = \text{tons } 0.84$$

Dead load per panel point, Fig. 186 = 6.3^{π} —

$$5' \times 0.84^{\pi} = \text{,, } 2.1$$

With FZ loaded @ 0.84^{π} /ft. run, the live

$$\text{and dead load reactions at } A \text{ are} = \text{,, } 5.91 \text{ and } 4.2$$

Panel loads at B are, live and dead

$$\text{respectively} = \text{,, } 1.18 \text{ and } 2.1$$

Hence maximum shear in the second panel

$$= (5.91 + 4.2)^{\pi} - (1.18 + 2.1)^{\pi} = \text{,, } 6.83$$

The point Z is known as the neutral point of the panel, because a load placed on the concrete floor at Z causes no live load shear whatsoever in this panel. To verify this place a 1 ton load at Z , then $\frac{3}{4}$ ton comes to the B stringer and $\frac{1}{4}$ ton to the C stringer, and

thence into the main beam. The live load reaction at A , taking moments at F , $= 1^T \times 18\frac{3}{4}' \div 25 = \frac{3}{4}^T$, and hence reaction F is $\frac{1}{4}^T$. Therefore, the live load shear in BC is reaction A — panel load $B = \frac{3}{4}^T - \frac{3}{4}^T = 0$, as stated. Neutral points only exist where the load arrives at the main girder through stringers, i.e., panel loading.

Similarly, a 1 ton load placed on the concrete at mid-span causes a local load at C and D of $\frac{1}{2}^T$, and a reaction at A and F of $\frac{1}{2}^T$. Shear from A to C is $\frac{1}{2}^T$ upwards, then at C the local $\frac{1}{2}$ ton load acts downwards, leaving no vertical force or shear in panel CD .

FISH-PLATES AND SPLICES

Fish Plates. Fig. 187, taken from a trader's hand book, is a typical illustration of a "standard" fish-plate. Fish-plates are

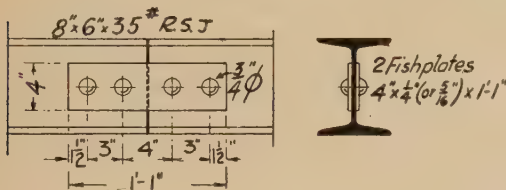


FIG. 187

useful for beam alignment, but in no way do they act as covers, as the following example clearly indicates.

Beam Splices can be designed :—

(1) By using only the net areas of the various parts of the beam in the calculations.

(2) By using the moment of inertia and the shear area of the beam in the calculations.

(3) So that the splice plates and rivets are sufficient to carry only the actual bending moment and shearing force occurring at the section.

(1) *The Area Method* develops the total net tensile strength of the beam, and the calculations employing this method are given in the fifth and sixth pages of Chapter VI., and refer to Fig. 188.

(2) *M. of I. Method.*

Gross *M. of I.* of beam (Fig. 188) = in.⁴ 115.1

Minus 4 holes in flanges, section *AA* =

$$4(0.648'' \times \frac{3}{4}'') (3.676)^2 = \text{,,} \quad 26.1$$

Net *M. of I.* of beam

$$= \text{,,} \quad \underline{89.0}$$

The gross Z of the beam is listed at $= \text{in.}^3 \quad 28.8$
 The net Z of the beam $= 89.0 \div 4'' = \text{,,} \quad 22.25$

Covers :—

Gross $M.$ of $I.$ of flange covers $= 2 @ 6'' \times \frac{11}{16}'' \times 4.344^2 = \text{in.}^4 \quad 155.3$

At AA , net $M.$ of $I.$ of flange covers $= \frac{3}{4} \times 155.3$ (as the holes are $\frac{1}{4}$ of the width) $= \text{,,} \quad 116.5$

At AA , gross $M.$ of $I.$ of web covers $= 2 @ \frac{1}{12} \times \frac{5}{16}'' \times 5\frac{1}{4}^3 = \text{,,} \quad 7.53$

Total moment of inertia of covers at section $AA = \text{,,} \quad 124.03$

$\therefore$ Net Z of covers at $AA = 124.03 \div 4\frac{11}{16}'' = \text{in.}^3 \quad 26.5$

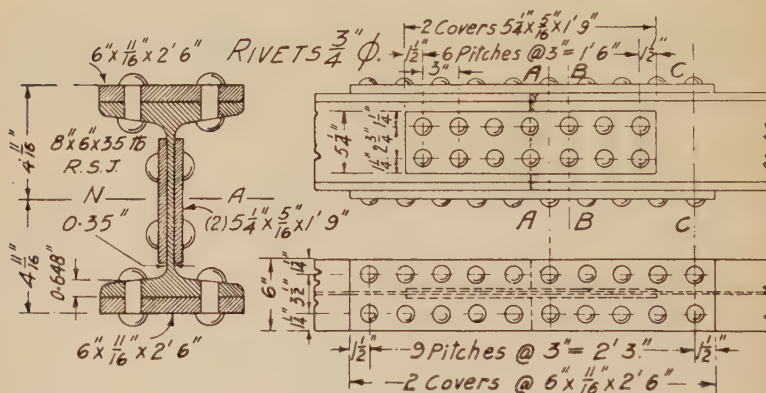


FIG. 188

Of the two sections of the beam alone, AA and BB , the former is the weaker, net flanges + gross web, and is, therefore, the actual strength of the beam which should be developed. The net modulus of the beam at CC equals that at AA , viz. a Z of 22.25 in.³ net.

The unholed beam can carry a maximum $B.M.$ of $ZF_t = 28.8F_t$, but only a bending moment of $22.25F_t$ when holed, so it follows that the splice should be situated at some position in the span where the bending moment does not exceed $22.25F_t$.

If a pitch less than 3 in. be used the net area of each flange will be less than the gross area — 2 rivet holes each, because of diagonal tearing.

Rivets :—

$M.$ of $I.$ of the ten rivets in each flange on one side of joint $= \text{shear area} \times \text{distance}^2 = 2 @ 10 \times 0.44 \times 4^2$, in terms of shear area $= \text{in.}^4 \quad 141$

In terms of transverse area, since $F_s = \frac{2}{3}F_t = \frac{2}{3}$ of 141 = in. ⁴	94
$M.$ of $I.$ of the six rivets in the web = bearing area $\times$ distance ² = 6 rivets @ $\frac{3}{4} \times 0.35 \times 1\frac{3}{8}$ ² , in	
terms of bearing area	= ,, 3
In terms of transverse area, since $F_b = \frac{4}{3}F_t = \frac{4}{3}$ of 3 = ,,	4
Equivalent $M.$ of $I.$ of rivets $\therefore$ = 94 + 4 = ,,	98

The net $M.$ of $I.$ of the beam, considering only minimum strength, was 89.0 in.⁴ The net $M.$ of $I.$ of the covers = 124.03 and the equivalent $M.$ of $I.$ of the rivets is 98, both in excess of the beam's net $M.$ of $I.$

The method of designing a splice "by areas" discussed in Chapter VI., and as illustrated by Fig. 188, is, therefore, entirely satisfactory when examined from the aspect of moment of inertia.

(3) *Actual B.M. and Shear Method.* Suppose the splice be placed, as it usually is, at a section where the bending moment is small, *e.g.*, say the bending moment at the desired position is $15F_t$, then there is no necessity to use heavy covers and riveting to develop a Z of 22.25, *i.e.*, a bending moment of $22.25F_t$. If it is permissible to curtail the flange plates when the bending moment decreases, surely the analogy holds good in that only the actually occurring bending moment should be developed, plus, say, an allowance of 10 per cent. = a total of $16.5F_t$. Similarly, the web covers could develop the actual shear at the section plus 5 per cent. excess, since the web covers, unlike the flange covers, are symmetrical. Such a course is often adopted with plate girders.

SPECIAL CASES OF BEAM LOADING

Bending in Two Planes. Approximate Solution, Fig. 189. As a common everyday example consider the case of an ordinary side or end framing angle horizontal carrying galvanized sheeting. Reference to Figs. 178 and 179, Vol. II., shows that the framing angle has to support two loads:—(1) The weight of the vertical sheeting which covers an area of 12 ft. 6 in. long by 6 ft. deep, *i.e.*, between horizontals.

(2) The external wind pressure acting in a horizontal plane. Let the intensity of the wind pressure be 20 lb. per square foot of vertical surface.

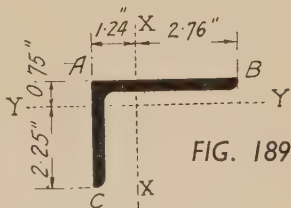


FIG. 189

Total weight of sheeting per angle of 12 ft. 6 in. span = W , vertically, estimated at, = ton	0.12
Total horizontal wind load P per angle = $12.5' \times 6' \times 20 \text{ lb./sq. ft.} \div 2240 =$ „	0.67
The minimum horizontal width of angle is limited to span $\div 40 = 150 \div 40 =$ in.	3.75

Either of two angles may be tried, (a) a $3\frac{1}{2}'' \times 3'' \times \frac{3}{8}''$, or (b) a $4'' \times 3'' \times \frac{5}{16}''$ with the longer leg in the plane of greater $B.M.$, i.e., the horizontal one. The second angle, possessing a larger modulus for less weight of steel, appears to be the more economical.

$M.$ of $I.$, $xx = 3.3.$	Z left	$= 3.3 \div 1.24 =$ in. ³	2.66
	Z right	$= 3.3 \div 2.76 =$ „	1.2
$M.$ of $I.$, $yy = 1.59.$	Z top	$= 1.59 \div 0.75 =$ „	2.12
	Z bot.	$= 1.59 \div 2.25 =$ „	0.71

Assuming, meantime, that the ends of the angle are freely supported, then :—

$$B.M._W \text{ in vertical plane, due to } W, \\ = Wl \div 8 = 0.12 \times 12.5 \times 12 \div 8 = \text{in.}^{\tau} \quad 2.25$$

$$B.M._P \text{ in horizontal plane, due to } P, \\ = Pl \div 8 = 0.67 \times 12.5 \times 12 \div 8 = \text{„} \quad 12.56$$

W causes compression (+) on the AB face
and tension (−) at C ;

while

P causes compression (+) on the AC face
and tension (−) at B .

Stresses at A .

$$B.M._W \div Z_t = 2.25 \div 2.12 = + \quad 1.06 \\ B.M._P \div Z_l = 12.56 \div 2.66 = + \quad 4.72 \quad = \tau/\text{sq. in.} \quad + 5.78$$

Stresses at B .

$$B.M._W \div Z_t = 2.25 \div 2.12 = + \quad 1.06 \\ B.M._P \div Z_r = 12.56 \div 1.2 = - \quad 10.46 \quad = \text{„} \quad - 9.40$$

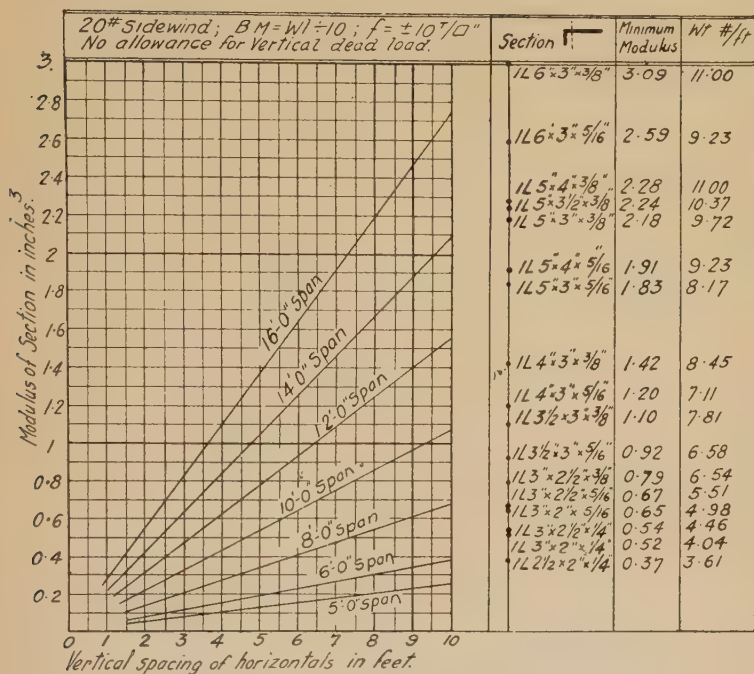
Stresses at C .

$$B.M._W \div Z_b = 2.25 \div 0.71 = - \quad 3.17 \\ B.M._P \div Z_l = 12.56 \div 2.66 = + \quad 4.72 \quad = \text{„} \quad + 1.55$$

The largest concentration of stress is fortunately a tensile one

at *B*. The compression face *AC* is securely held by the vertical sheeting against any lateral movement in that plane.

Had the horizontal framing member been a joist, with the web horizontal, it would have had its principal axes parallel to both planes of loading, and the foregoing treatment would, except for



HORIZONTAL FRAMING ANGLES FOR SHEETING

FIG. 190

This graph may also be used for any intensity of wind pressure P by multiplying the value of the modulus given on the graph by $P \div 20$; e.g., if $P = 15$ lb/sq. ft., span = 16 ft., and vertical spacing is 8 ft., then the modulus required is $15/20$ of the graph value of $2.2 = 1.65$ in.³. A suitable section is one angle $5'' \times 3'' \times \frac{5}{16}''$ and $Z = 1.83$ in.³.

the slight eccentricity of W acting on one face, have been free from reproach. Since the angle, however, is a non-symmetrical section the stresses found are very approximate.

Current Design of Framing Angles uses $Pl \div 10$ for the bending moment instead of $Pl \div 8$ and entirely neglects the effects of the vertical dead load W . Experience has shown this to be quite satisfactory, and possibly the reasons for this are :—

(1) The sheeting is firmly bolted to the angles and forms with them one solid and massive section, wherein the sheeting serves as the compression flange of a "wind girder," and the outstanding angle legs act as ribs carrying tension. The moment of inertia of the combined elements must be much larger than that of the unaided angles. Further, because of the continuous sheeting, these horizontal angles have a certain degree of end continuity or fixity against the wind.

(2) With buildings of moderate height, say up to 40 ft. or thereby, the assumed wind pressure may never be reached owing to the frictional drag of the ground upon the wind gust.

(3) The combined sheeting probably acts as a very deep vertical girder, and is thus able to carry its own weight from column to column without stressing the horizontal angles, at least to any appreciable extent.

Wind loads occur frequently but seldom at maximum value. When one of the loads on a member is a wind load an increase of 25 per cent. may be permitted in the working stress; for a tension member, F_t becomes $\frac{5}{4}F_t$. Thus dead load + live load ($D + L$, respectively) require an area $A_1 = (D + L) \div F_t$. When the wind load (W) is added the required area $A_2 = (D + L + W) \div \frac{5}{4}F_t$ and the area adopted is the greater area, either A_1 or A_2 .

As suggested, using $Pl \div 10$ for the bending moment due to wind and neglecting the dead load, the stress at B is as follows.

$$\begin{aligned} \text{Max. } B.M. \text{ due to wind} &= 0.67 \times 12.5 \times 12 \div 10 = \text{in. tons} \quad 10.05 \\ \text{Wind stress at } B &= M \div Z = 10.05 \div 1.2 = \tau/\text{sq. in.} \quad - 8.4 \end{aligned}$$

The graph given herewith (Fig. 190) is based upon current practice.

The immediately preceding remarks do not vitiate the example of bending in two planes, which is complete in itself.

Non-symmetrical Bending. In the examples on floor beams the plane of loading coincided with one of the principal axes of the section and the neutral axis was at mid-depth. With the framing angle (Fig. 191), the horizontal wind is parallel to YY , and therefore inclined to both the principal axes UU and VV .

The British Standards Institution, in its complete lists of dimensions and properties of angles, gives the following values for a $4" \times 3" \times 0.3"$ angle. Area = 2.011 sq. in., $M.$ of $I_x = 3.185$, $I_y = 1.535$, $I_u = 3.894$, $I_v = 0.825$ and $\tan \alpha = 0.548$ ($\alpha = 28.7^\circ$). Investigating only the action of the wind, the bending moment carried by the angle is 12.56 in. tons, i.e., using $Pl \div 8$ for the purposes of comparison.

Resolve the bending moment into two components parallel to the principal axes UU and VV , and then find the fibre stress at B due

to each considered separately, from the standard formula of $f = \frac{My}{I}$.

The algebraic sum of these stresses gives the fibre stress at B . Refer to Fig. 192.

Component $u = P \sin \alpha = 12.56 \times \sin 28.7^\circ = \text{in. tons} \quad 6.03$

Component $v = P \cos \alpha = 12.56 \times \cos 28.7^\circ = \text{,,} \quad 11.02$

About UU axis $f(\text{tension}) =$

$$\frac{My}{I} = \frac{vy}{I_u} = \frac{11.02 \times 2.6}{3.894} = \text{tons/sq. in.} - 7.36$$

About VV axis $f(\text{tension}) =$

$$\frac{My}{I} = \frac{uy}{I_v} = \frac{6.03 \times 0.842}{0.825} = \text{,,} - 6.15$$

Total stress at B is (tension) $= \text{,,} - 13.51$

The values I_u and I_v may be obtained from the abridged list of sections given in Table 2, Appendix. Minimum radius of gyration =

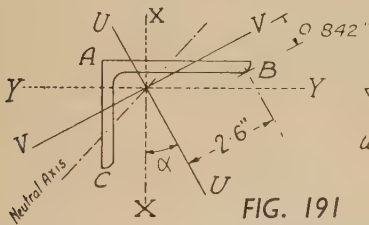


FIG. 191

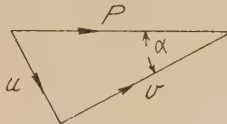


FIG. 192

0.64". $\therefore$ minimum M . of I . = area $\times$ radius² = $2.01 \times 0.64^2 = 0.823$. Also I polar = $I_x + I_y = I_u + I_v$, hence $I_u = I_x + I_y - I_v = 3.185 + 1.535 - 0.823 = 3.89 \text{ in.}^4$

It will be noted that a $4" \times 3" \times 0.3"$ and not a $4" \times 3" \times \frac{5}{16}"$ (0.31") is considered; the difference in thickness of 0.01 in. may be neglected. The approximate method using the XX axis for the wind gave an extreme fibre stress of -10.46 tons per square inch; an error of almost 23 per cent. Despite this large discrepancy, practice always uses the approximate method except for important members.

Stresses at the other corners of the angle can be found in a similar manner.

The foregoing problem can also be solved graphically by using the momental ellipse of which VV is the major and UU the minor axis. This method requires more time, and is not so accurate, as it entails the drawing, or at least part, of an ellipse; however, it is a

useful check, and students are referred to text-books on Strength of Materials for the mode of procedure.

Three different sets of stresses for such a simple element of a structure as a framing angle are apt to be bewildering. The particular case of a framing angle was chosen to exemplify one of the many apparent divergences between theory and practice. Usually the reason for this difference hinges on the assumptions made by the designer; *e.g.*, either in the assumed value of the loads, in their distribution, or in the aid afforded by adjoining members. A brief *résumé* may clear the difficulty.

(1) Bending in two planes.—The method is approximate when applied to a non-symmetrical section. In the case discussed it was assumed that the angle was free to deflect in any direction; an assumption which is incorrect.

(2) Bending in one plane, $B.M. = Pl \div 10$ as given by the graph.—This method is empirical and indicates that practice recognizes that the angle obtains aid from the sheeting and from the end cleats attaching the angle to the columns.

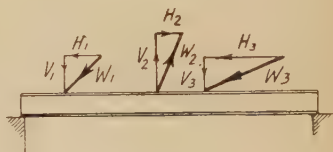


FIG. 193

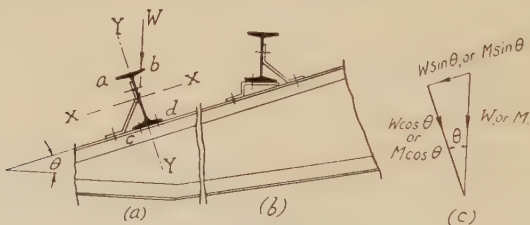


FIG. 194

(3) The method given under the heading of non-symmetrical bending is the correct one to employ in place of (1) when the section is non-symmetrical. As with the approximate method of (1) the angle was assumed free to deflect in any direction, an assumption which has a large influence upon the resulting calculated stresses.

The Load Inclined to the Neutral Axis but Acting in the Plane of the Web of the Beam. Fig. 193 illustrates a girder with three such inclined loads, W_1 , W_2 and W_3 .

Method. Resolve the inclined loads into their vertical and horizontal components and treat the problem after the method given in Chap. IV., Vol. II., on "Combined Direct and Lateral Thrust."

The Load Inclined to the Plane of the Web.

Fig. 194 shows the nose of a plated cantilever girder in the gallery of a theatre. The load W of Fig. (a) may, with small error, be assumed to pass through the centre of gravity of the joist. Now resolve either W or, more simply, the vertical bending moment M , occasioned by W , into two directions parallel to the principal axes XX and YY . The component $M \cos \theta$ creates compressive stresses along the ab face and tensile stresses along cd . Similarly, $M \sin \theta$ produces compression at b and d and tension at a and c .

$$\therefore \text{Stress at } a = + M \cos \theta \div Z_x, - M \sin \theta \div Z_y.$$

$$\text{Stress at } b = + M \cos \theta \div Z_x, + M \sin \theta \div Z_y.$$

$$\text{Stress at } c = - M \cos \theta \div Z_x, - M \sin \theta \div Z_y.$$

$$\text{Stress at } d = - M \cos \theta \div Z_x, + M \sin \theta \div Z_y.$$

Maximal stresses occur at b and c .

Detail (b) eliminates the non-symmetrical bending by using a $\frac{5}{8}$ in. thick, or thereby, M.S. bent plate or chair. The bent plate cleat to the web serves as a safeguard against overturning.

CHAPTER XI

DESIGN OF A JOIST AND CHANNEL CRANE GANTRY GIRDER

IN modern shop equipment much depends upon the crane installation. Rapidity of handling loads is essential towards shop output, and crane design has kept pace with the demands made upon it.

The steelwork which carries these quick-acting cranes must be heavier than the steelwork which supports slow-moving cranes. With quick-acting electric overhead travelling ("E.O.T.") cranes the stresses in the gantry girders are produced almost instantaneously, whereas with slow-moving hand-operated cranes the bending stresses in the girder are induced gradually from zero up to their maximal values, as the crane traverses the girder from the end towards the centre.

To allow for this, and taking into consideration that F_t is taken at 10^7 /sq. in., an impact factor varying from 25 per cent. to 30 per cent. should be added to all the live load stresses caused by E.O.T. cranes. For hand-operated cranes an increase of 10 per cent. on the live load stresses should be ample. Stresses due to speeding and braking, and also crane mismanagement, are more liable to occur in a long, wide workshop than in one which is cramped, and thus the 30 per cent. impact factor should be reserved for the longer gantry. Should the cranes be working in a heavy foundry where loads up to (or occasionally over) the full lift capacity are continually encountered it would be advisable to specify the impact allowance of 30 per cent., but for ordinary shops where the loads vary from light to heavy an impact addition of 25 per cent. appears to be quite reasonable and in consonance with world practice. For vertical loads the *B.S. 449 - Buildings* specifies an impact allowance of 25 per cent. for all E.O.T. cranes and 10 per cent. for hand-operated cranes.

Permissible or working stresses vary somewhat with designers. Some firms in designing crane girders use a working tensile stress of 6 to 7.5 $\times 10^7$ /sq. in. for the combined dead and live load stresses without any impact factor. Where the dead load stresses are accurately known there is no apparent reason why the working stress should not

be 10π /sq. in. as for ordinary work. The live loads could then be associated with impact factors which would vary with the nature of the loading. However, it is not necessary to enter into polemics on this subject. In the following example F_t will be taken at 10π /sq. in. in conjunction with an impact allowance of 25 per cent.

Comparing the two above working stresses for a tensile live load flange force of W^T :—

Method adopted.

Required area = $(W + 25\%) \div 10 = 0.125W$ net sq. in.

Other method.

Required area = $W \div (\text{from 6 to } 7.5) = 0.16W \text{ to } 0.133W$ net sq. in.

Longitudinal Forces on Crane Girders Fig. 195. The largest of these, especially in quick-acting electric overhead travelling cranes, is due to the sudden application of the brakes. The frictional resistance to the sliding of the locked wheels upon the rail is supplied by the crane girder. This element in turn distributes it amongst all the crane column shafts.

The coefficient of friction μ for steel sliding on steel is variously stated at 0.12 and upwards. Thus the longitudinal forces on the gantry track of two lines of girders may be assumed to be at least 12 per cent. of the total weight of crane and its lift load.

As a particular example :—A large-span E.O.T. crane of 40 tons lift weighs 32 tons without its load. The total load carried by the gantry girders is 72 tons, and hence the total maximum longitudinal thrust along the girders is 12 per cent. of 72 tons, *i.e.*, about 8.6 tons ; or, assuming that each gantry girder takes an equal share, it is about 4.3 tons per girder.

The above implies that the load is in mid-shop, whereas if the load lifted be adjacent to one side of the building, that pair of wheels nearer the load will carry 28 tons each, depending upon the width of shop, instead of 18 tons each ($72 \div 4$ wheels), and the longitudinal thrust on this girder, theoretically, may rise to 2 (wheels) $\times 28 \times \mu$, *i.e.*, 6.72 tons.

This force will be carried from the rail surface through the rivets or bolts connecting the rail to the top flange. These are usually spaced at a reeled pitch of about 1 ft., giving a total of 41 or thereby in a 40-ft. span gantry girder. The shear load per fastening is $6.72 \div 41 = 0.16$ ton, so that these fastenings are safe. No further notice is taken of this thrust in designing the crane girders ; but it should be taken into consideration when designing the columns and other parts of the structure.

Consider this question from another aspect. Say the speed of longitudinal or main travel is 250 ft. per minute and that the crane

can brake itself in a length of 10 ft. To find the retarding force supplied by the gantry girder :—

$$v^2 = u^2 + 2as. \quad \text{Where } v = \text{final velocity in feet per second} = 0$$

$$\therefore 0 = (4\frac{1}{6})^2 - 2a \times 10 \quad u = \text{initial velocity in feet per second} = 250 \text{ ft. per min.} \\ = 4\frac{1}{6} \text{ ft. per sec.}$$

$$\text{whence } a = 0.87$$

a = acceleration in feet per second per second ; when braking it is retardation or negative.

s = space travelled with brakes on before coming to rest = 10 ft.

Force = mass $\times$ acceleration. Assuming that the retarding force is constant (which it is not) over this 10-ft. length, then Force

$$= \frac{72 \times 2240 \times 0.87}{g} \text{ lb., or in tons} = \frac{72 \times 0.87}{32.2} = 2^r \text{ nearly.}$$

This retarding force, on the pair of gantry girders, rises to 3 tons when the speed is increased to 300 ft. per minute ; specifications naturally reflect this wide divergence of values. The *B.S. 449—Building* specifies 5 per cent. of the actual wheel loads for both electric and hand-operated cranes. One American specification assumes longitudinal forces, induced by starting and stopping the crane, to be 10 per cent. of the wheel loads.

Rolling friction will cause also a longitudinal thrust through the main gantry track, but its value is much smaller than the braking thrust.

Experimental results on friction are highly conflicting and the difficulty of assessing thrusts will be seen from the following table. Table A contains the results of Captain Galton and Mr. Westinghouse, and are for steel wheels sliding on steel rails. M. Poirée, experimenting on the Paris and Lyons Railroad with a four-wheeled waggon, whose brakes were screwed up so that the wheels skidded, found the results given in Table B.

A		B	
Speed in miles/hour.	μ	Speed in miles/hour.	μ
10	0.110	9—14	0.208
15	0.087	14—18	0.179
25	0.080	18—20	0.167
38	0.047	30—40	0.144

Although both agree that the coefficient decreases with an increase of speed, its values as found by M. Poirée are twice those as found by Messrs Galton and Westinghouse.

Lateral Forces on Crane Girders (Fig. 195) may be caused by :—

(a) The thrust due to the sudden stopping of the crab and load when traversing the crab girders.

(b) The crane dragging weights across the shop floor.

The foregoing cases cannot occur simultaneously.

In (a), as with the longitudinal gantry girders, the frictional resistance of the rail is transferred into the crab girders and from them into the crosshead girders, thence, as point loads through the main wheels, into the top or compression flanges of the gantry girders.

The positions of the main wheels when maximum lateral bending

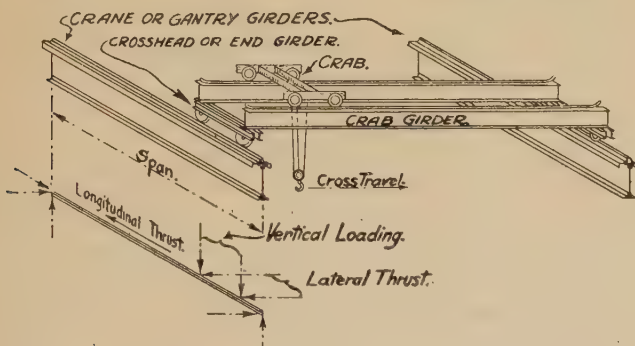


FIG. 195

and shear take place on the gantry girder will be the same as those when maximum vertical bending moment and shear occur.

(b) The crane is often requisitioned to drag weights across the shop floor. If the load is extremely massive it is usually mounted on roughly fashioned rollers, probably running on a timber plank track. The lateral thrust and pull on the compressive flanges of the gantry girders then become a matter of conjecture. The resisting forces are, firstly, the friction of the main wheel treads upon the gantry rails and, secondly, the forces offered by the flanges of the main wheels bearing against the gantry rails.

The double flanged steel wheels of the end carriages have their flanges and treads machined and radiused to suit the longitudinal rails upon which they run. Furthermore, the cranes are so designed that in the unlikely event of a derailment they cannot fall into the shop.

Because of the unknown factors in the horizontal transverse loading there has been quite a diversity of opinion as to the allowance which should be made for it in the design calculations. Current specifications show more unanimity than formerly, but even yet the divergence of opinion is sometimes large. Thus the *B.S. 449—Building* specifies that the total lateral thrust, shared equally between the two gantry girders, is to be taken as 10 per cent. of the combined weights of crab and lift load. One American specification states that the value of the lateral thrust to be applied at each end carriage, *i.e.*, on one gantry girder, as 10 per cent. of the load lifted (or a total of 20 per cent. of the lift load for the gantry track of two girders). As a general rule the weight of the crab is approximately one-fifth of the lift load and thus the American loading is two-thirds larger than the British one.

End Carriage Wheel Base. The distance apart of the wheels on the gantry girder is a function of the crane span across the shop. The *B.S. 466—E.O.T. Cranes* specifies the following :—

Crane span in ft. .	Up to 60	60 to 72	72 and up
Minimum wheel base .	Span $\div$ 5	12 ft.	Span $\div$ 6

Summary. *Vertical Loads.* Increase by an impact factor *I* of 25 per cent. for E.O.T. and 10 per cent. for hand-operated cranes.

Lateral Thrust. The total value, to be shared equally by the two gantry girders, is 10 per cent. of the combined weights of crab and lift load for E.O.T. cranes and 5 per cent. for hand-operated ones.

Longitudinal force on the gantry girders is 5 per cent. of the actual wheel loads for both electric and hand-operated cranes.

Flange Width is usually not less than a twenty-fifth of the span. The upper flange is a compression member and may fail sideways, especially under side or lateral thrust. The outer edges of the flanges are stiffened by having a continuous angle riveted to them or, if the crane is light, the upper flange is composed of an inverted channel as in Fig. 196. Using the deepest channel rolled, *viz.*, 17" $\times$ 4" $\times$ 44 lb., the limiting span for this type is in the neighbourhood of 35 ft. The narrower the flange width is in proportion to the span the smaller is the working stress (see plate girder flanges, Chapter I of Vol. II.). When the flange plates are not stiffened by edge angles the working stress is further lowered.

A 5 tons E.O.T. crane has been running for many years now on gantry girders of 20" $\times$ 7½" $\times$ 89 lb. R. S. Joists. The unsupported span is 32 ft., so that the ratio of flange breadth to span works out at the alarming figure of 1 $\div$ 51.

EXPLANATORY TEXT

DESIGN OF A 20-FT. SPAN GANTRY GIRDER

Dead Load, item 1. The live load bending moment was calculated as in item 4, and—neglecting dead load meantime—this divided by the working tensile stress of 10 tons per square inch gives an approximation to the required modulus. Thus $784 \div 10 =$ a modulus of about 78. The rules of items 10 and 11 were then considered, from which was evolved the section of Fig. 196. Allowing for a rail of 45 lb. per yard, the dead weight works out at 0.8 of a ton.

Crane Rails. A locomotive rail supported on sleepers—spaced at 2 ft. 6 in. centres or thereby—is permitted to carry a wheel load of 1 ton for every 10 lb. per yard weight of rail. On this basis the Pacific Type of Locomotive, which has a 10 tons wheel load, would require a 100 lb. per yard rail. Crane rails, on the other hand, are supported throughout their entire length; there is no bridge or beam action between sleepers, and in consequence the rail may be permitted to carry a heavier running load. Practice would apparently assign a safe wheel load of 3 tons for every 10 lb. per yard of “bridge rail,” and for special “flat-bottomed” rails a wheel load of $3\frac{1}{2}$ tons to 4 tons per 10 lb. per yard of rail.

The rail is assumed to offer no help to the crane girder in carrying the loads.

Weight of Crab is usually about one-fifth of the maximum load lifted plus $\frac{1}{2}$ ton. Use this rule if actual weight is unknown.

Item 2. The reaction will be a maximum just previous to the leading wheel passing off the span. Live load reaction *A* is found by taking moments about end *B*.

Item 4 and Fig. 198. “Maximum bending moment, under any wheel, occurs when the centre line of the span is midway between the centre of gravity (C.G.) of the load system and that wheel.” Both wheels have the same load and their joint C.G. therefore lies midway between them. The position for maximum bending moment thus occurs when the centre line (C.L.) of the span lies between the wheels and is at a distance of a quarter of the wheel base from either wheel. Live load reaction *B* is found by taking moments about *A*.

If the wheel base is greater than 0.586 the effective span, maximum bending moment will occur when one wheel is off the span and the remaining wheel at mid-span.

Maximum Dead Load B.M. takes place at mid-span and is equal to $Wl \div 8$. The bending moment parabola changes its shape but slightly in the near neighbourhood of the centre line, so that the *B.M.* at *X* is approximately that at mid-span. The error of this assumption is small and is on the right or full side.

CALCULATIONS

DESIGN OF A 20-FT. SPAN GANTRY GIRDER (FIG. 196)

Loading, etc. One E.O.T. crane of 10 tons capacity. Maximum load on each end carriage wheel is 9.7 tons, and occurs when the crab and the load are adjacent to one main gantry rail.

Effective Span. The length of the girder (*i.e.*, c/c of columns along the length of the shop) is 20 ft. Allowing that the girder has a 6 in. length of bearing on each column bracket support, the effective span is 19 ft. 6 in., *i.e.*, centre of bearing to centre of bearing.

Shop width, c/c of columns	=	53' 0"	
Deduct $2(5'' + 9\frac{1}{2}'')$, Fig. 125	=	2' 5"	
Crane span, c/c of rails	=	50' 7"	
Wheel base = $50' 7'' \div 5$	=		10'

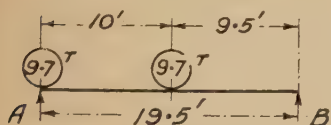


FIG. 197

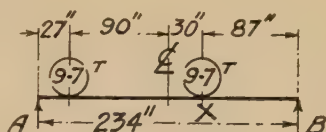


FIG. 198

Working Stresses, B.S. 449 (p. xiv)

Dead Load of Girder and rail is estimated at ton 0.8 1

Weight of Crab = $(10^T \div 5) + \frac{1}{2}^T$ = tons 2.5

Maximum Vertical Shear. Fig. 197.

Live load reaction A =

$9.7^T (9.5' + 19.5') \div 19.5 =$ „ 14.4 2

Impact, add 25% = „ 3.6

Total *L.L.* + *I.* = „ 18.0

Dead load reaction $A = 0.8 \div 2 =$ „ 0.4

Total end shear *L.L.* + *I.* + *D.L.* = tons 18.4 3

Maximum Vertical B.M. Fig. 198.

Live load reaction B =

$9.7^T (27'' + 147'') \div 234'' = 7.21$ tons.

Maximum *B.M.* @ X , *L.L.* =

$R_B \times 87'' = 7.21^T \times 87'' =$ in. tons 627 4

Impact, add 25% = „ 157

Total *L.L.* + *I.* = „ 784

Dead load maximum *B.M.* @

mid-span = $0.8^T \times 234'' \div 8 =$ „ 23

Total maximum *B.M.* @ X is

approximately = in. tons 807 5

Items 6, 7 and 8. Fig. 197 gives the position of the wheels for maximum horizontal shear (due to lateral thrust) as well as the position of loads for maximum vertical shear. The distances are the same, the only difference being in the planes of loading and values of the loads. Hence item 7 is obtained directly from item 2 by multiplying the latter by the ratio of the loads, *viz.*, $0.31^T \div 9.7^T$. Similarly, item 8 = item 4 $\times (0.31^T \div 9.7^T)$.

Position of N.A. The centroid of the channel's cross-sectional area is given in the list of properties of sections as 0.78 in. from the heel. Then the channel area multiplied by the distance of its centroid from the base, plus the R.S.J. area multiplied by its distance of centroid, gives the total moment of area about the base. This figure divided by the total area gives the position of the joint centre of gravity and, therefore, of the neutral axis. No deductions are necessary on account of rivet holes, as these occur in the compression flange. The rivet fills the hole, and so is assumed to carry the thrust or compression in the top flange fibres from one side of the hole to the other.

Moment of Inertia, or more correctly, second moment, about any line parallel to that through the centroid of the area = *M.* of *I.* about the centroidal line plus the area of the section multiplied by the square of the distance between the parallel lines of reference, *i.e.*, *M.* of $I_1 = M.$ of $I_0 + Ad^2$ as given in text-books on Mechanics.

It is immaterial whether the centroid of the component part be above or below the neutral axis of the combined section, *M.* of *I.* is always the sum. Algebraically *d* may be positive or negative with reference to the *xx* axis, but its square is always positive. The inertias and areas of the sections are taken from the tabulated list of properties of sections.

Item 17. Section modulus = *M.* of *I.* $\div$ distance from N.A. to the fibre considered and is in inches³ (inches⁴ $\div$ inches). The outermost fibres have the greatest stress, and since these are situated at unequal distances from the N.A. there will be a modulus for the top fibres and another for the bottom fibres. It appears rather puerile to give the foregoing, but inertias and moduli always provide a trap for beginners.

Item 18. The lateral thrust is assumed to act in the plane of the centre of gravity of the upper flange. Acting as it does at rail level, it has really a lever arm producing torque. This small lever arm and, therefore, the torque are neglected. No help is assumed to be afforded by the lower or tensile flange in resisting lateral thrust. However, should this help be considered, then the torque—due to the thrust multiplied by the distance from the line of action of the

Lateral Force =

10% of $(2.5^x + 10^x)$	= tons	1.25	
per wheel = $1.25 \div 4$ wheels	=	ton 0.31	6
Lateral Maximum Shear =			
$14.4^x(0.31^x \div 9.7^x)$	=	„ 0.46	7
Lateral Maximum B.M. =			
$627^x(0.31^x \div 9.7^x)$	=	in. tons 20	8
Longitudinal Force on girder is 5%			
of $9.7^x \times 2$ wheels	=	ton 0.97	9
Girder Depth varies from span $\div 10$,			
to span $\div 12$, i.e., from		in. 23 to 19	10
Flange Width is span $\div 25$	=	„ 9	11
Modulus required = Z =			
$B.M. \div F_t = 807 \div 10 =$		in. ³ 81	12

The foregoing three items tentatively fix the section as being composed of an 18" $\times$ 6" $\times$ 55 lb. R.S.J. ($Z = 93.5$ in.³) with a 9" $\times$ 3" $\times$ 17.5 lb. channel riveted to the upper flange.

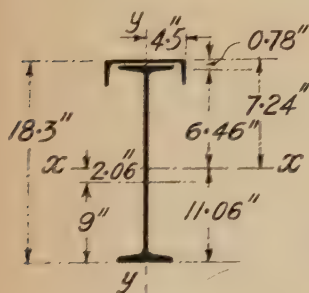


FIG. 199

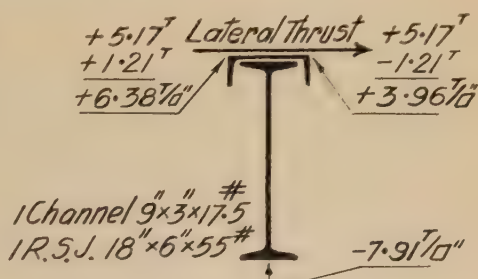


FIG. 200

Position of Neutral Axis, etc.

Section.	Area, sq. in.	Lever Arm, in.	Moment, in. ³	
R.S.J.	16.18	9	145.62	
Channel	5.14	18.3 - 4.78	90.05	
	21.32	$\times$ 11.06	= 235.67	13

Moment of Inertia, axis xx (Appendix, Tables 4 and 5)

Section.	I , Self, in. ⁴	A , in. ²	D , in.	AD^2 , in. ⁴	I , total, in. ⁴	
R.S.J.	841.8	16.18	2.06	68.66	910.46	
Channel	3.75	5.14	6.46	214.50	218.25	
	845.55	plus		283.16	= 1128.71	14

thrust to the N.A. (xx in this case)—should also be taken into consideration.

Items 19 and 20. When the upper, or compression, flange is well supported laterally throughout its length then $F_c = 10$ tons per square inch. The fewer the number of lateral supports the greater becomes the lateral flexibility and F_c is no longer 10 but becomes steadily smaller in value. Thus, as with a column, F_c depends upon the slenderness ratio of length $\div$ breadth or, more correctly, length $\div$ radius of gyration, *i.e.*, l/k (both in inches). Hence the working stress for F_c given on p. xiv has two values, *viz.*, the lesser of either 10 tons per square inch or 8.33 tons as obtained from the formula of item 20. The joist and channel section is not symmetrical about the xx axis and thus K_1 has the value of unity.

Items 21 to 22. Due to vertical loading, the upper flange is in compression with the extreme top lamina of 9 in. width stressed to 5.17 tons per square inch. In addition, assuming that the lateral thrust comes from the left, the horizontal loading causes further compression in the left channel leg, while the right channel leg tends to be in tension; the upper flange of the girder deflecting in a horizontal plane, concave side towards the left hand and convex side towards the right. The stresses are added together as shown on Fig. 200.

Item 23 would apparently indicate that the section is under stressed. On looking at the list of joists it will be seen that the next lightest joist is 16" $\times$ 6" $\times$ 50 lb., a saving of 5 lb. per foot over the 18-in. joist. The ratio of depth to span, however, falls from $\frac{1}{13}$ to $\frac{1}{14.5}$, and therefore deflection will be larger with the lighter joist. If the 9-in. channel be replaced by an 8-in. channel the ratio of flange width to span drops from $\frac{1}{26}$ to $\frac{1}{30}$, with a consequent lowering of the working compressive stress (item 20). Further, from Table 4, the 9-in. channel possesses a clear distance between root fillets of 7 in., which is ample for the 6-in. flange of the joist; on the other hand, the 8" $\times$ 3" channel has 6 in. exactly, and therefore no allowance is given for sectional growth of the joist flange.

The calculated section will therefore be used. Details are given on Fig. 196.

Item 24. The breadth of section b may be taken as 6 in., the flange width of the joist, or the channel width of 9 in.; it is immaterial which value is adopted as the breadth cancels out in item 25. A maximum pitch of 16*t* is permitted by B.S. 449.

Fish-plates are used for the purpose of alignment, the thickness shown on the drawing being adopted without calculation. When fitting these, the $\frac{13}{16}$ in. diameter holes are placed, say, at the right

Moment of Inertia, axis yy

R.S.J. (Table 5, yy) 23.64

Channel (Table 4, xx) 62.52

86.16 15

M. of I., top flange, axis yy $= 62.52 + \frac{1}{2} \text{ of } 23.64 =$ in.⁴ 74.3 16*Rad. of Gyration = $\sqrt{I/A}$* $= k_y = \sqrt{(86.16 \div 21.32)} =$ in. 2.0*Section Modulus = $I/y = Z$* 17 $Z_{x'} \text{ top} = 1128.7 \div 7.24 =$ in.³ 156 $Z_{x'} \text{ bot.} = 1128.7 \div 11.06 =$ " 102 $Z_{y'} \text{ top fl. only} = 74.3 \div 4.5 =$ " 16.5 18**Working Stresses** from p. xiv $F_t, (- \text{ sign}) =$ τ/sq. in. 10 $F_c, (+ \text{ sign}) \text{ either} = 10$ " 20or $= 1000 K_1 \div l/k_y, \text{ i.e.}$ $F_c = 1000 \times 1 \div (20 \times$ 12 $\div 2) =$ " 8.33 $F_w \text{ on web} =$ " 6.5**Actual Stresses**Vertical loads, $F_t = M \div Z_b = 807 \div 102 =$ " -7.91 21 $F_c = M \div Z_t = 807 \div 156 =$ " +5.17Lateral loads, $F_t = F_c = M \div Z = 20 \div 16.5 =$ " ± 1.21 22Top fl. stresses $= +5.17 + 1.21 =$ " +6.38 23and $= +5.17 - 1.21 =$ " +3.96Vertical shear area $= 18" \times 0.42" =$ sq. in. 7.56" " stress $= 18.4 \tau \div 7.56 =$ τ/sq. in. 2.43Lateral shear area $= 9" \times 0.3" =$ sq. in. 2.7" " stress $= 0.46 \tau \div 2.7 =$ τ/sq. in. 0.2

Riveting. The intensity "*q*" of the longitudinal shear between joist flange and channel, where they have a common area of contact, is given by the formula (see Chapter I, Volume II)

$q = \frac{SG}{Ib}$. Where *S* = vertical shear at the point = 18.4τ, item 3.

I = moment of inertia at the section = 1128.7 in.⁴, item 14.

G = first moment about the N.A. of the area above the plane considered = 5.14 × 6.46 in.³

b = breadth of section at the point considered = width of joist flange = 6 in.

end of girder and the $\frac{7}{8}$ in. diameter holes at the left end. The latter are equivalent to, but cheaper than, slotted holes. They permit of a longitudinal adjustment of the girder, as also does the $\frac{1}{8}$ in. clearance between the ends of the girders.

Transverse adjustment is obtained by reamering out the holes in the crane column cap and also the common holes in the horizontal legs of the zed-shaped cleat between column and channel flanges. The crane girders must be exactly parallel and true to line before being permanently bolted up to the main structure. The $9\frac{1}{2}$ in. from the rail centre line to the face of the roof shaft is net ; should there be rivet heads on the roof shaft, then the extra $\frac{1}{2}$ in. or more must be added to the $9\frac{1}{2}$ in. dimension.

Temperature. The expansion due to temperature is seldom provided for in this country with crane girders. If expansion becomes large the girders will move the bolts in the " slotted " fish-plates.

Riveting will be closest at the end of span.

$$q = \frac{18.4\tau (5.14 \times 6.46 \text{ in.}^3)}{1128.7 \text{ in.}^4 \times 6 \text{ in.}} = \tau/\text{sq. in. } 0.09 \quad 24$$

$$\therefore \text{Total shear per inch run of span for a 6 in. width of flange} = 0.09 \times 6 = 0.54\tau \quad 25$$

$$\text{Total shear per foot run of span for a 6 in. width of flange} = 0.54 \times 12 = 6.48\tau$$

Using $\frac{3}{4}$ in. diameter rivets.

$$\text{Value of one in single shear} = 0.44 \times 6 = 2.64\tau$$

$$\text{and in bearing on channel web} = \frac{3}{4}'' \times 0.3 \times 12 = 2.7\tau$$

$$\text{Number of rivets required per foot} = 6.48\tau \div 2.64\tau = 3$$

$$\text{Maximum pitch for compression member rivets} = 16t = 16 \times 0.3 = \text{in. } 4.8$$

Adopt $4\frac{1}{2}$ in. pitch throughout, see Fig. 196.

$$\text{Number of rivets per foot} = 5.3$$

Stiffeners.

$$\text{Ratio of web thickness to joist depth} = 0.42 \div 18 = \frac{1}{43}$$

No stiffeners required, since these are only necessary when the ratio is less than $\frac{1}{75}$.

Rail joint at 1 ft. from the end of the girder causes less jarring than if it were at mid-span, or even at the column centre line where the crane girders butt end to end.

REFERENCES

BRITISH STANDARD SPECIFICATIONS

(Published by the British Standards Institution, 28 Victoria Street, London, S.W.1)

No. 449. *The Use of Structural Steel in Building.*

No. 466. *Electric Overhead Travelling Cranes.*

CHAPTER XII

DESIGN OF A WORKSHOP GALLERY

EXPLANATORY TEXT

A GALLERY, suitable for light work, is to be attached to the existing columns of a workshop. In consequence the columns will be subjected to an increase in the direct load carried and also to an additional bending moment. It is assumed that investigation has shown these additions permissible.

Item 1. From the Table of "Floor Live Loads" given in Chapter V of Volume II the loading for this floor is the dead load plus either a 60 lb. per square foot live load for all the floor or, for the individual floor boards only, a uniformly distributed live load (independent of span) of 480 lb. on a one-foot width of boarding.

Item 2. The planking (tongued and grooved) may be of any suitable width, but a 12 in. width is considered for calculation purposes; its weight is 14 lb., giving a total *U.D.L.* of 494 lb.

Item 3. Timber calculations assume all spans to be simply supported. This gives larger bending moments than that obtained by taking continuity and end-fixing moments into account. Local imperfections, knots, etc., make this course advisable.

Item 4. The actual fibre stresses are satisfactory, being under the permissible. For timber protected from the weather, the permissible transverse fibre stress may vary from 1000 to over 1400 lb. per square inch, depending upon the grade of timber.

Item 5. The shear parallel to the grain must not exceed 120 lb. per square inch. In a rectangular section the horizontal shear along the grain at the neutral axis is $1\frac{1}{2}$ times the mean vertical shear (as explained in Chapter I, Volume II).

Items 6 and 7. *B.S. 449—Building* permits a deflection of $\text{span} \div 325$. The well known rule of $\frac{1}{40}$ in. per foot of span, *i.e.*, $\text{span} \div 480$, is therefore still usable (and is sometimes desirable). Recall that the actual sizes of a timber section may be less than the nominal (or so called) sizes because of planing and sawing. Where rapid surface wear is anticipated a scantling larger than the calculated nominal one can be adopted with advantage.

Item 8. On the final drawing the stringer span is slightly less than 12 ft. 6 in. (safe, no recalculation necessary). The mid stringer

CALCULATIONS

Working stresses, B.S. 449 (p. xiv).

Loading, either live for all floor	= lb./sq. ft.	60	1
or boarding only, a total <i>U.D.L.</i>	= lb. of 14 +	480	2

Boarding. Try 1" T. & G. (too thin).

Dead wt. on 48" free span, 1' wide
 = $4' \times 1' \times \frac{1}{12}' @ 42 \text{ lb./cu. ft.}$
 = 14 lb.

Live load = $4' \times 1' @ 60 \text{ lb./sq. ft.}$
 = 240 lb.

Total load (less than item 2) = lb. 254

Hence max. *B.M.* = $Wl/8 = 494 \times 48/8$ = in. lb. 2964 3

Z of 12" width = $\frac{1}{8}bd^2 = \frac{1}{8} \times 12 \times 1^2$ = in.³ 2

Max. fibre stress = $M/Z = 2964/2$ = lb./sq. in. ± 1482

" " " permissible = " ± 1000

Try $1\frac{1}{2}$ " tk. flooring

$Z = \frac{1}{8}bd^2 = \frac{1}{8} \times 12 \times (1\frac{1}{2})^2$ = in.³ 4.5

$I = \frac{1}{12}bd^3 = \frac{1}{12} \times 12 \times (1\frac{1}{2})^3$ = in.⁴ 3.38

Max. fibre stress = $M/Z = 2964/4.5$ = lb./sq. in. ± 659 4

End vert. shear = $\frac{1}{2}$ of $494 \div (12 \times 1\frac{1}{2})$ = " 14

End horiz. " = $1\frac{1}{2} \times 14$ = " 21 5

Deflection $\Delta = \frac{5 Wl^3}{384 EI}$

= $\frac{5 \times 494 \text{ lb.} \times 48^3 \text{ in.}^3}{384 \times 1,500,000 \text{ lb./sq. in.} \times 3.38 \text{ in.}^4}$ = in. 0.14

Permissible Δ = span $\div 325 = 48" \div 325$ = in. 0.15 6

Adopt flooring $1\frac{1}{2}$ " thick 7

Stringers, 12' 6" span @ 4' 0" c/c. 8

Mid stringer, area supported

= $4' \times 12.5'$ = sq. ft. 50

Dead load, plank = $50 (1\frac{1}{2} \div 12) @ 42 \text{ lb.}$ = lb. 262 9

Steel stringer, say 12.5' @ 10 lb./ft. = " 125 10

Nailing strip, say (3/12) $(2\frac{1}{2}/12) \times$
 $12.5 @ 42 \text{ lb.}$ = " 27 11

414

Live load = 50 sq. ft. @ 60 lb./sq. ft. = " 3000 12

Total load say = 3400

= tons 1.51

End reaction = 1.51×2 = ton 0.75

Total *B.M.* on stringer = $Wl \div 8$

= $1.51 \times 150" \div 8$ = in.¹ 28.3

Z required = $28.3 \div 10$ = in.³ 2.83

supports twice the load carried by either of the external ones, but the depth of all three must be the same.

Items 9, 11 and 12 are known and their sum is 3289 lb. or $1\frac{1}{2}\pi$. Hence the approximate $B.M. = 1.5\pi \times 150'' \div 8 = 28.1$ in. π . Z required $= M \div F_t = 28.1 \div 10 = 2.81$ in. π . An angle, joist or channel with this modulus weighs about 10 lb. per foot; hence item 10. The channel is adopted as being the most suitable section.

Item 13. Placing a coach screw about 3 in. from each end of the stringer and at 3 ft. pitch thereafter gives 5 in the 12 ft. 6 in. length—more than ample as shown by the shear strength (black bolts, p. xv).

Item 14. Half the weight of one bracket frame is given to the top mid-panel point (same reason as in item 8).

Item 15. The forces acting in the members of the frame may be obtained by two methods, "Section and Moments" and "Stress Diagram." Both methods will be given, although only one is necessary.

In Fig. 201 (a) the frame is cut by section line xx . By the third law of statics ($\Sigma M = 0$) the sum of the moments of the forces about any point, say b , must sum to zero, since that part of the frame shown in heavy line is at rest. Taking moments about point b , (M_b), then 0.4π at $a \times 4$ ft. to b can only be countered by the unknown force in the cut bar $ac \times$ perpendicular lever arm of 1.79 ft. The unknown forces in bars ab and cb need not be considered because, passing through the moment centre b , they have no lever arms and therefore have no moments about point b . The nose a tends to rotate clockwise round b as centre, so tending to close the cut in bar ac , i.e., the force in ac is compression (plus sign).

Item 15 (b) and Fig. 201 (b). The "wanted" bar is ab and the moment centre is c . The nose a tending to rotate about c , as centre, opens the cut (tension). $M_c : -0.4\pi \times 4$ ft. $= ab \times 2$ ft.

Item 15 (c) and Fig. 201 (c). The force in bd must be the same as that in ab since the other two forces at b are at right angles to abd and therefore have no horizontal components to add to ab and bd . Moment equation (c) proves this. For a similar reason the force in cb must be 0.8π which is in line with it and which compresses it. As a check moments can be taken about point a with section xx cutting bars bd , bc and ac .

Item 15 (d) and Fig. 201 (d). Following the rule of "Take a section to cut three bars, including the wanted bar, and take moments at the point where the two unwanted bars meet," gives xx as the section line and a as the moment centre. The cut structure shown in heavy line tends to rotate clockwise round moment centre a thus opening the cut in cd , which is therefore under tension.

Section :—

1 L, $5'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$	@ 13.6 lb.	Z = in. ³	2.93
1 R.S.J.	$4'' \times 3'' \times 10$ lb.	Z = „	3.89
Adopt 1 Chan.	$5'' \times 2\frac{1}{2}'' \times 10.2$ lb.	Z = „	4.75
„	„	I = in. ⁴	11.87
Shear area of channel	= $5'' \times 0.25''$	= sq. in.	1.25
Shear stress on „	= $0.75^{\tau} \div 1.25$	= $\tau/\text{sq. in.}$	0.6

$$\Delta = \frac{5 \times Wl^3}{384 \times EI}$$

$$= \frac{5 \times 1.51 \pi \times 150^3 \text{ in.}^3}{384 \times 13,000 \pi / \text{sq. in.} \times 11.87 \text{ in.}^4} = \text{in.} \quad 0.43$$

$$\Delta_{\text{permissible}} = \text{span} \div 325 = 150'' \div 325 = \text{,,} \quad 0.46$$

Coach screws, $\frac{7}{16}$ " dia. by $2\frac{1}{4}$ " long under head ; 5 off per 12' 6" span. Shear value = 5×0.15 sq. in. @ 4 τ /sq. in. = tons 3.0 13

Nailing strip, 3" wide $\times$ 2½" deep.

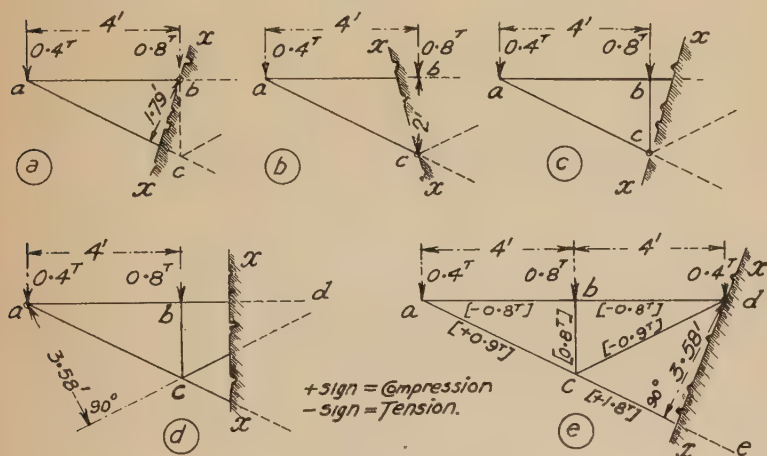


FIG. 201

Braced Bracket.

Two braced frames form one bracket, *i.e.*, each frame carries the reactions of the 12' 6" span stringer channels.

Dead weight of one frame. Allowing for
gussets say $(8' + 9' + 2' + 4.5')$ length
of angles @ 5 lb./ft. = 117 lb. = ton

0.05 14

The method of Section and Moments is more fully explained in the author's book on *Influence Lines*, Chapter IV.

Item 15 (e) and Fig. 201 (e) not only give the force in bar ce but also the reaction of the column on the bracket at point e . The bar ce carries an axial load (no bending moment) and since "action and reaction are equal and opposite" the push of the column on the bracket at e must be along ec .

Figs. 202 and 203, Stress Diagram. It is not necessary to find the reactions before commencing the stress diagram for a cantilever frame. These may be obtained, however, as follows. In the present example the 0.4τ load adjoining the column immediately enters the column and does not affect the cantilever. The resultant of the 0.4τ at the nose and the mid-panel load of 0.8τ is 1.2τ at X , 16 in. to the left of the mid panel. It has just been shown that the reaction at the base of the bracket travels upwards along $3 : A$ to X . The third, and remaining, force acting on the structure must be along the line from X to the upper right-hand column connection at $C : 3$. (The body is in equilibrium and therefore the three external forces must pass through one point, X .) In Fig. 202 (b), ac is 1.2τ ; $c : 3$ (the reaction at the top of the bracket) is 1.7τ ; and $3 : a$ (the thrust reaction at the bracket base) equals the force in member $3 : A$ of 1.8τ .

Bow's Notation. Place a capital letter in the space between adjoining external forces and do so in a *clockwise* direction. Within the frame itself place numbers, again proceeding in a *clockwise* direction. Draw the load line to scale and parallel to the loads and use small or lower case letters; thus from a to b is downwards 0.4τ , then follows the next force $B : C$ represented by $b : c$, 0.8τ in length on the load scale.

The directions of all three forces acting at nose $A : B : 1$ are known together with the magnitude of one force, $A : B$, hence the two unknown values can be obtained from the triangle of forces of Fig. 203 (a). Through a draw a line parallel to $1 : A$ and through b draw a line parallel to $B : 1$. These intersect at point 1 and the force in member $1 : A$ is represented by $1 : a = 0.9\tau$, while $B : 1$ is $b : 1 = 0.8\tau$. These can be measured on the diagram since the load scale is given. These three forces act at the nose $A : B : 1$ and with reference to this point, $A : B = a : b$ downwards; $B : 1 = b : 1$ eastwards, see arrow, i.e., away from the nose or pulling the nose eastwards, which means that this member could be a rope and thus the member is under tension. Bar $1 : A = 1 : a$ pushes the nose north-west (see arrow) and to push requires a rigid bar (not a rope) and so this member is in compression.

Load at mid-panel point = $\frac{1}{2}$ (stringer
load + weight of bracket)

$$= \frac{1}{2}(1.51^T + 0.05^T) = \text{ton}$$

0.8

Load at nose of cantilever = $\frac{1}{2}$ of $0.8^T =$ „

0.4

Forces by Section and Moments. Fig. 201.

(a), M_b ; $0.4^T \times 4' \div 1.79'$,

cut closes $\therefore ac =$ „

+ 0.9 15

(b), M_c ; $0.4^T \times 4' \div 2'$,

cut opens $\therefore ab =$ „

- 0.8

(c), M_c ; $(0.4^T \times 4' + 0.8^T \times 0') \div 2'$,

cut opens $\therefore bd =$ „

- 0.8

(d), M_a ; $(0.4^T \times 0' + 0.8^T \times 4') \div 3.58'$,

cut opens $\therefore cd =$ „

- 0.9

(e), M_a ; $(0.4^T \times 8' + 0.8^T \times 4') \div 3.58'$,

cut closes $\therefore ce =$ „

+ 1.8

(f), „ $cb =$ „

+ 0.8

(Compression = +, and tension = - sign.)

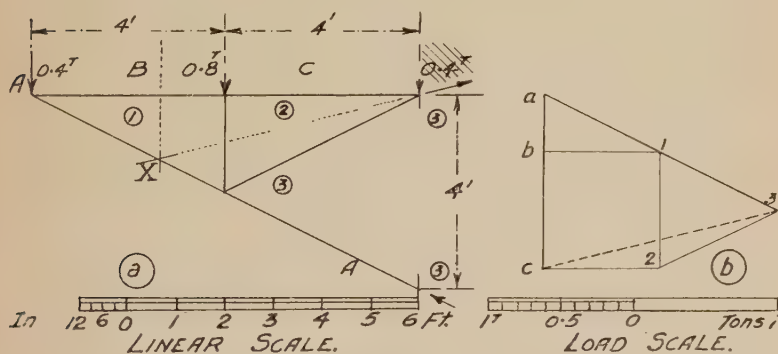


FIG. 202

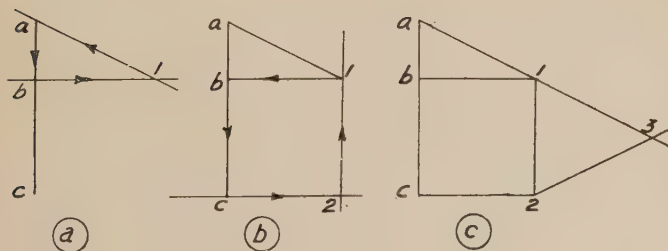


FIG. 203

Fig. 203 (b) shows the forces at the mid-panel point. Here there are four forces, two known and two unknown. The known forces $1 : B$ (not $B : 1$, because the lettering must be read clockwise round the panel point) and $B : C$ appear in the stress diagram as $1 : b$ and bc . Through c draw $c : 2$ parallel to $C : 2$ and through 1 draw $2 : 1$ parallel to bar $2 : 1$. These two lines intersect at 2 , so defining the magnitudes of the two unknowns. Reading clockwise round diagram 203 (b) gives b to c southwards, c to 2 eastwards away from panel point and so bar $C : 2$ is like a rope pulling the panel point eastwards, *i.e.*, the member is in tension (-0.8τ); then follows from 2 due north to 1 , which means that the bar $2 : 1$ pushes the panel point upwards (which cannot be done by a rope) and hence this bar is in compression ($+0.8\tau$). Finally, from 1 due west to b means that bar $1 : B$ pulls the panel point westwards and so is under tension as previously found (-0.8τ).

This last sentence is apparently at variance with that made in the course of considering the nose and Fig. 203 (a), when it was stated that bar $B : 1$ (note, it was not termed $1 : B$ then) pulled the nose towards the east. Both statements are in agreement, for, if this member be made of an elastic rope, it will tend to pull one panel point towards the other, *i.e.*, the member is in tension as stated.

The lower mid-panel point near X is next considered. Here again there are only two unknowns, $2 : 3$ and $3 : A$, which in stress diagram (c) intersect at 3 , thus defining their magnitudes. Bar $2 : 3$ (not $3 : 2$) is away from the point (north-east, stress diagram (c)) and thus this member is in tension; while $3 : a$ acts north-west or towards the panel point and so member $3 : A$ is under compression. Had this panel point been attempted immediately after the nose panel there would have been three unknowns—thus no solution is possible until the mid-panel point stress diagram is completed to provide the second known force $1 : 2$.

The forces are written alongside the respective bars in Fig. 201 (e).

Item 16. A $2\frac{1}{2}$ in. leg is the narrowest angle leg which will take a $\frac{3}{4}$ in. dia. bolt or rivet. Since both legs are riveted the minimum size for tie abd therefore becomes $2\frac{1}{2}$ in. $\times$ $2\frac{1}{2}$ in. $\times$ the minimum structural thickness of $\frac{1}{4}$ in. The method of finding the net area of an angle tie is explained in the text to Figs. 135 and 136.

Item 17. The outstanding leg of this tie is not riveted and metal is saved by making this leg 2 in. instead of $2\frac{1}{2}$ in. The upper end of this bar is slightly lowered from the position indicated on the structural diagrams of Figs. 201 (e) and 202 (a) in order to rivet it to the column shaft. This will cause but little change in the force in this bar.

Item 18. Only one leg is riveted and so the minimum section is used, as in the previous item. It is always the gross cross-sectional area of a strut which is considered, provided that the holes are rivet filled.

When an angle strut is tested to destruction it folds along the zz axis (see the diagram accompanying Table 2 of the Appendix) since the minimum radius of gyration is about this axis. The slenderness ratio of a strut is the length (in inches) divided by the radius of gyration (also in inches). The greater this ratio the smaller becomes the safe end load.

Most strut formulæ consider the standard case of a pin-ended strut, *e.g.*, an angle with a single rivet at each end. The length, L , of this strut supporting an end load W is the distance between the end rivets. Using two rivets per end considerably increases the possible length of the column for the same end load W . The equivalent pin-ended length of the longer column is thus 0.8 times its actual length, see Fig. 205. This explanation is possibly over simplified but it will suffice for this design, which is preliminary to those given in Vol. II. Among the various tables of safe loads for struts given in Vol. II is one which applies only to discontinuous angle struts as shown on Fig. 205. For all slenderness ratios between 0 and 100 the tabulated values of the end loads (in tons per square inch and termed $F_c/2$ for this particular set) are given by the straight line equation accompanying this illustration. In fact it is easier to use this equation than the tabulated values with the necessary interpolations.

Item 19. The end fastenings of strut *ce* provide more rigidity than do the relatively thin gusset plates of Fig. 205 and strut *bc*. At end *e* the angle is riveted to the rigid column and at end *c* it is continuous and not discontinuous. This strut could thus safely carry a larger load than that given by the formula of Fig. 205, which, however, will be again used. By using the tie plate on the underside of the strut at the bottom mid-panel point the slenderness ratio is halved, otherwise the 3 in. $\times$ 3 in. angle would be free to deflect outwards from the plane of the paper for its full length from nose to column.

This member is that nearest the main shop floor and a side blow would have serious results, especially since it is a strut. For this reason and to give greater rigidity a heavier section was employed.

For the reasons given in the text to Fig. 137 the frame centre lines of Fig. 204 are the rivet lines.

Item 20. A load leaves a gusset plate and enters an angle through the common rivets—travels the length of the angle and leaves it through the connecting rivets to enter the remaining gusset plate.

Design of Members. Forces are given on Fig. 201 (e).

Tie abd (continuous) Max. force = — 0.8 π 16

The Tables mentioned are in the Appendix.

Adopt 1L 2 $\frac{1}{2}$ " $\times$ 2 $\frac{1}{2}$ " $\times$ $\frac{1}{4}$ " (Table 1)

Area = sq. in. 1.18

Less rivet hole & $\frac{1}{2}$ out. leg (Table 10) = „ 0.47

Net area, see Fig. to Table 10 = „ 0.71

Actual $f_t = -0.8 \div 0.71 = \pi/\text{sq. in.} - 1.13$

Permissible F_t , p. xiv = „ - 9.0

Tie cd Max. force = — 0.9 17

Adopt 1L 2 $\frac{1}{2}$ " $\times$ 2" $\times$ $\frac{1}{4}$ " (Table 2)

Area = sq. in. 1.06

Less rivet hole & $\frac{1}{2}$ out. leg (Table 10) = „ 0.41

Net area, see Fig. to Table 10 = „ 0.65

Actual $f_t = -0.9 \div 0.65 = \pi/\text{sq. in.} - 1.4$

Permissible F_t , p. xiv = „ - 9.0

Short strut bc Max. force = + 0.8 π 18

Adopt 1L 2 $\frac{1}{2}$ " $\times$ 2" $\times$ $\frac{1}{4}$ " (Table 2)

Area = sq. in. 1.06

Min. rad. gyration, k_z (Table 2) = in. 0.42

Length L , c/c end rivet groups = „ 16

Length $l = 0.8L = „ 13$

$l/k = 13 \div 0.42 = „ 31$

Permissible stress $F_e 2 = 6 - 0.029 l/k$

$= 6 - 0.029 \times 31 = \pi/\text{sq. in.} 5.1$

Actual stress $f_e 2 = +0.8 \div 1.06 = „ 0.8$

Main strut ce (and *ac*) Max. force = + 1.8 19

Adopt 1L 3" $\times$ 3" $\times$ $\frac{1}{4}$ " (Table 1)

Area = sq. in. 1.44

Min. rad. gyration, k_z (Table 1) = in. 0.59

Length L , c/c end rivet groups = „ 56

Length $l = 0.8L = „ 45$

$l/k = 45 \div 0.59 = „ 76$

Permissible stress $F_e 2 = 6 - 0.029 l/k$

$= 6 - 0.029 \times 76 = \pi/\text{sq. in.} 3.8$

Actual stress $f_e 2 = +1.8 \div 1.44 = „ 1.25$

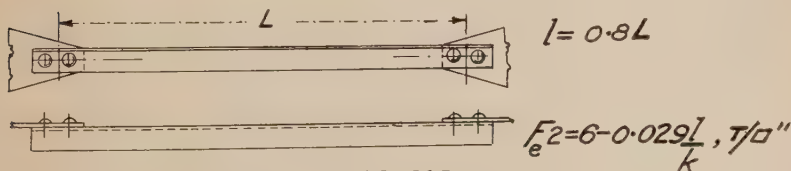


FIG. 205

The connecting rivets must therefore carry the load in single shear and bearing. It was stated in the text concerning Figs. 135 and 136, that never less than two rivets are given at the end of an angle.

At c in member ae the load to the left of c is 0.9τ and to the right of c it has increased to 1.8τ . The gusset plate at c is the only place where this increment of 0.9τ can come and therefore sufficient rivets must be placed through angle and gusset plate to permit this increment to enter the main strut at c .

Item 21. The side pressure of workmen leaning against the top tube of the handrail is estimated at 25 lb. per foot run of rail. This value is ample since no crowding will occur in an 8 ft. wide gallery. Every vertical angle supports about 4 ft. of rail and is thus subjected to a total horizontal load outwards of 100 lb. The distance of the top tube to the centre of the channel web is 42 in. and so the bending moment on the two connecting bolts (vertical angle to channel web) is 4,200 in. lb. These two web bolts provide a resisting couple whose lever arm is 2 in. The area at the base of the thread of a bolt may be found by the empirical rule given at the end of Chapter IV, alternatively, consult p. xv.

The angle vertical also carries the same bending moment of 4,200 in. lb.

The top tubing is continuous for the full length of the gallery and so the maximum bending moment is not $Wl \div 8$ but $Wl \div 12$ and occurs at the supports (vertical angles). The external dia. and the bore of the mild steel steam tubing are given together with the formula for the section modulus.

Rivets, $\frac{3}{4}$ " finished dia. Value of one in		20
$S.S.=2.65^T$; in $\frac{1}{4}$ " $B=2.25^T$ (p. xv)		
Fig. 201 (e). No. of rivets reqd. at :—		
a and d of ad also		
b and c of bc is $0.8^T \div 2.25^T$	=	1
c and d of cd also		
a of ae is $0.9^T \div 2.25^T$	=	1
e of ae is $1.8^T \div 2.25^T$	=	1
c of ae is $(1.8^T - 0.9^T) \div 2.25^T$		1
Handrailing. Lateral thrust, top rail	= lb./ft.	25
Load per standard, <i>i.e.</i> , on 4' length	= lb.	100
Lever arm to <i>C.G.</i> channel web bolts	= in.	42
<i>B.M.</i> thereon = 100 lb. $\times$ 42"	= in. lb.	4200
Load/chan. bolt = 4200 $\div$ 2" apart	= lb.	2100
Base of thread area, $\frac{5}{8}$ " dia. bolt	= sq. in.	0.2
Permis. load = 0.2 @ 5 ^T /sq. in., p. xv	= lb.	2240
<i>B.M.</i> , angle standard ; 4200 in. lb.	= in. ^T	1.9
Z_x , 1L 3" $\times$ 2 $\frac{1}{2}$ " $\times$ $\frac{1}{4}$ " (Table 2)	= in. ³	0.54
Result. stress = $M/Z = 1.9 \div 0.54$	= ^T /sq. in.	$\pm$ 3.5
<i>B.M.</i> on tube = $Wl/12 = 100 \times 48/12$	= in. lb.	400
	= in. ^T	0.18
Z of tube (extern. $D=1\frac{11}{32}$ " ; intern. $d=1$ ")		
$= \pi(D^4 - d^4) \div 32D$	= in. ³	0.16
Result. stress = $M/Z = 0.18 \div 0.16$	= ^T /sq. in.	$\pm$ 1.1
Batten and tie plates. Thickness		
= span between rivets $\div$ 50		
= $14\frac{3}{4}$ " $\div$ 50, see Chap. IV, Vol. II	= in.	$\frac{5}{16}$

APPENDIX

(FORMERLY VOLUME III)

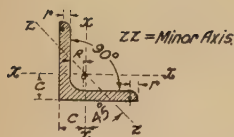
TABLES

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* *Extracted by permission of the British Standards Institution from their reports "No. 4—1932. DIMENSIONS AND PROPERTIES OF CHANNELS AND BEAMS" and "No. 4A—1934. DIMENSIONS AND PROPERTIES OF EQUAL ANGLES, UNEQUAL ANGLES AND TEE BARS," official copies of which may be obtained from the Secretary of the Institution, 28, Victoria Street, London, S.W.1.*

TABLE 1

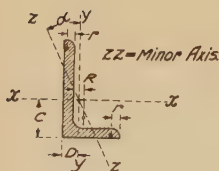
EQUAL ANGLES



Notes.—The larger the number of stars in Column 1, the more common is the section. Sections can be obtained in thicknesses varying by $\frac{1}{16}$ in., e.g., $4" \times 4" \times \frac{5}{16}"$, etc., although these are not listed. The properties of such sections may be obtained, with sufficient exactitude, by interpolation.

See Notes.	Size.	Area sq. in.	Weight per ft. lb.	Radii.		Dimen- sion, C.	Moment of Inertia xx.	Mini- mum Section modulus xx.	Radii of Gyration.	
				Root R.	Toe r.				xx.	Least zz.
**	$2 \times 2 \times \frac{1}{8}$	0.94	3.19	.24	.17	0.58	0.34	0.24	0.60	0.39
		1.15	3.92	"	"	0.61	0.40	0.29	0.59	0.38
		1.36	4.62	"	"	0.63	0.47	0.34	0.59	0.38
*	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{8}$	1.06	3.61	.26	.18	0.64	0.49	0.30	0.68	0.44
		1.31	4.45	"	"	0.67	0.59	0.37	0.67	0.43
		1.55	5.26	"	"	0.69	0.69	0.44	0.67	0.43
***	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{16}$	1.18	4.04	.27	.19	0.70	0.68	0.38	0.76	0.49
		1.46	4.98	"	"	0.73	0.83	0.47	0.75	0.48
		1.73	5.90	"	"	0.75	0.96	0.55	0.74	0.48
***	$3 \times 3 \times \frac{1}{8}$	1.44	4.89	.30	.21	0.83	1.20	0.55	0.91	0.59
		1.78	6.04	"	"	0.85	1.47	0.68	0.91	0.58
		2.11	7.17	"	"	0.88	1.72	0.81	0.90	0.58
		2.75	9.35	"	"	0.92	2.18	1.05	0.89	0.58
***	$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{5}{16}$	2.09	7.11	.33	.23	0.97	2.38	0.94	1.07	0.68
		2.49	8.45	"	"	1.00	2.80	1.12	1.06	0.68
		3.25	11.05	"	"	1.05	3.57	1.46	1.05	0.68
**	$4 \times 4 \times \frac{5}{16}$	2.40	8.17	.36	.25	1.10	3.61	1.24	1.23	0.78
		2.86	9.73	"	"	1.12	4.26	1.48	1.22	0.78
		3.75	12.75	"	"	1.17	5.46	1.93	1.21	0.78
*	$4\frac{1}{2} \times 4\frac{1}{2} \times \frac{5}{16}$	2.72	9.24	.39	.27	1.22	5.21	1.59	1.38	0.89
		3.24	11.00	"	"	1.24	6.15	1.89	1.38	0.88
		4.25	14.45	"	"	1.29	7.92	2.47	1.37	0.88
**	$5 \times 5 \times \frac{5}{16}$	3.61	12.28	.42	.29	1.37	8.53	2.35	1.54	0.98
		4.75	16.16	"	"	1.42	11.04	3.08	1.52	0.98
		5.86	19.93	"	"	1.47	13.37	3.78	1.51	0.97
***	$6 \times 6 \times \frac{5}{16}$	4.36	14.82	.48	.34	1.61	14.95	3.40	1.85	1.18
		5.75	19.55	"	"	1.66	19.48	4.49	1.84	1.18
		7.11	24.17	"	"	1.71	23.73	5.54	1.83	1.17
		8.44	28.69	"	"	1.76	27.74	6.54	1.81	1.17
		9.73	31.10	"	"	1.81	31.51	7.51	1.80	1.16
**	$7 \times 7 \times \frac{5}{16}$	6.75	22.95	.54	.38	1.91	31.42	6.17	2.16	1.38
		8.36	28.42	"	"	1.96	38.45	7.63	2.14	1.37
		9.94	33.79	"	"	2.01	45.12	9.04	2.13	1.37
		11.48	39.05	"	"	2.06	51.45	10.41	2.12	1.36
		13.00	44.20	"	"	2.10	57.46	11.73	2.10	1.36
*	$8 \times 8 \times \frac{5}{16}$	9.61	32.68	0.60	.42	2.20	58.26	10.05	2.46	1.57
		11.44	38.89	"	"	2.25	68.58	11.94	2.45	1.57
		13.24	45.00	"	"	2.30	78.44	13.77	2.43	1.56
		15.00	51.01	"	"	2.35	87.85	15.55	2.42	1.56

TABLE 2
UNEQUAL ANGLES

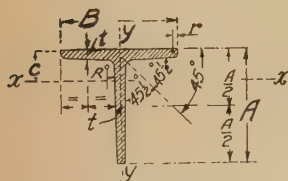


Notes.—The larger the number of stars in Column 1, the more common is the section. Sections can be obtained in thicknesses varying by $\frac{1}{16}$ in., e.g., $4" \times 3" \times \frac{1}{8}"$, although these are not listed. The properties of such sections may be obtained, with sufficient exactitude, by interpolation.

See Notes.	Size, inches.	Area in sq. in.	Wt. per ft. lb.	Radii, inches.		Distances to C. of G.		Moments of Inertia about		Section Moduli about		Radii of Gyration about			Tan α .
				Root R.	Toe r.	C.	D.	xx.	yy.	xx.	yy.	xx.	yy.	zz.	
•	$2\frac{1}{2} \times 2 \times \frac{1}{4}$	1.06	3.61	.26	.18	0.77	0.53	0.63	0.36	0.37	0.24	0.77	0.58	.42	.62
•		1.31	4.45	"	"	0.80	0.55	0.77	0.43	0.45	0.30	0.77	0.57	.42	.62
••	$3 \times 2 \times \frac{1}{8}$	1.55	5.26	"	"	0.82	0.57	0.89	0.50	0.53	0.35	0.76	0.57	.41	.61
••		1.19	4.04	.27	.19	0.98	0.48	1.06	0.38	0.52	0.25	0.94	0.56	.43	.43
••		1.46	4.98	"	"	1.00	0.51	1.29	0.45	0.65	0.30	0.94	0.56	.43	.43
••		1.73	5.90	"	"	1.03	0.53	1.50	0.53	0.76	0.36	0.93	0.55	.42	.42
•••	$3 \times 2\frac{1}{2} \times \frac{1}{4}$	1.31	4.47	.29	.20	0.89	0.65	1.14	0.72	0.54	0.39	0.93	0.74	.52	.68
•••		1.62	5.51	"	"	0.92	0.67	1.39	0.87	0.67	0.48	0.93	0.73	.52	.68
•••		1.92	6.54	"	"	0.94	0.70	1.62	1.02	0.79	0.56	0.92	0.73	.52	.67
•••	$3\frac{1}{2} \times 3 \times \frac{1}{8}$	1.56	5.32	.32	.22	1.01	0.77	1.86	1.26	0.75	0.56	1.09	0.90	.62	.72
•••		1.93	6.58	"	"	1.04	0.79	2.27	1.54	0.92	0.70	1.08	0.89	.62	.72
•••		2.30	7.81	"	"	1.07	0.82	2.67	1.80	1.10	0.83	1.08	0.88	.62	.72
••		3.00	10.20	"	"	1.12	0.87	3.40	2.28	1.43	1.07	1.06	0.87	.61	.71
•••	$4 \times 3 \times \frac{1}{8}$	2.09	7.11	.33	.23	1.24	0.75	3.30	1.59	1.20	0.71	1.26	0.87	.64	.55
•••		2.49	8.45	"	"	1.27	0.77	3.89	1.87	1.42	0.84	1.25	0.87	.64	.55
•••		3.25	11.05	"	"	1.32	0.82	4.97	2.37	1.85	1.09	1.24	0.85	.63	.54
•••	$4 \times 3\frac{1}{2} \times \frac{1}{8}$	2.67	9.09	.35	.24	1.19	0.94	4.09	2.91	1.45	1.14	1.24	1.04	.72	.75
••		3.50	11.91	"	"	1.24	0.99	5.24	3.72	1.90	1.48	1.22	1.03	.72	.75
•		4.30	14.61	"	"	1.29	1.04	6.28	4.45	2.31	1.81	1.21	1.02	.71	.74
•••	$5 \times 3 \times \frac{1}{8}$	2.86	9.73	.36	.25	1.68	0.69	7.25	1.97	2.18	0.85	1.59	0.83	.65	.36
•••		3.75	12.75	"	"	1.73	0.74	9.33	2.51	2.86	1.11	1.58	0.82	.64	.35
•••		4.18	14.23	"	"	1.76	0.77	10.31	2.76	3.18	1.24	1.57	0.81	.64	.35
•••	$5 \times 3\frac{1}{2} \times \frac{1}{8}$	3.05	10.37	.38	.26	1.59	0.85	7.63	3.09	2.24	1.16	1.58	1.01	.75	.48
•••		4.00	13.61	"	"	1.64	0.90	9.84	3.96	2.93	1.52	1.57	1.00	.75	.48
•		4.92	16.74	"	"	1.69	0.94	11.89	4.74	3.59	1.86	1.55	0.98	.74	.47
•••	$5 \times 4 \times \frac{1}{8}$	3.24	11.00	.39	.27	1.51	1.01	7.97	4.53	2.28	1.52	1.57	1.18	.85	.63
•••		4.25	14.45	"	"	1.56	1.06	10.29	5.83	2.99	1.98	1.56	1.17	.84	.62
•••		5.24	17.80	"	"	1.61	1.11	12.44	7.02	3.67	2.43	1.54	1.16	.84	.62
•••	$6 \times 3 \times \frac{1}{8}$	2.72	9.24	.39	.27	2.09	0.61	10.13	1.74	2.59	0.73	1.93	0.80	.64	.26
•••		3.24	11.00	"	"	2.12	0.63	11.99	2.05	3.09	0.87	1.93	0.80	.64	.26
•••		4.25	14.45	"	"	2.17	0.68	15.51	2.62	4.05	1.13	1.91	0.78	.63	.26
••		5.24	17.80	"	"	2.22	0.73	18.80	3.14	4.98	1.38	1.89	0.77	.63	.25
•••	$6 \times 3\frac{1}{2} \times \frac{1}{8}$	3.42	11.63	.41	.29	2.01	0.77	12.62	3.21	3.16	1.18	1.92	0.97	.76	.34
•••		4.50	15.30	"	"	2.06	0.82	16.36	4.13	4.15	1.54	1.91	0.96	.75	.34
•••		5.55	18.86	"	"	2.11	0.87	19.85	4.96	5.11	1.89	1.89	0.95	.75	.33
•••	$6 \times 4 \times \frac{1}{8}$	3.61	12.28	.42	.29	1.91	0.92	13.21	4.74	3.23	1.54	1.91	1.15	.87	.44
•••		4.75	16.16	"	"	1.97	0.97	17.14	6.11	4.25	2.02	1.90	1.13	.86	.44
•••		5.86	19.93	"	"	2.02	1.02	20.82	7.37	5.23	2.48	1.88	1.12	.86	.43
•	$7 \times 3\frac{1}{2} \times \frac{1}{8}$	3.80	12.91	.44	.31	2.44	0.71	19.28	3.31	4.23	1.19	2.25	0.93	.75	.26
•		5.00	17.00	"	"	2.50	0.76	25.07	4.27	5.57	1.56	2.24	0.92	.74	.26
•		6.17	20.99	"	"	2.55	0.81	30.53	5.14	6.86	1.91	2.22	0.91	.74	.26
•	$7 \times 4 \times \frac{1}{8}$	5.25	17.85	.45	.32	2.39	0.90	26.26	6.33	5.70	2.04	2.24	1.10	.86	.33
•	(up to)	7.69	26.14	"	"	2.49	1.00	37.42	8.86	8.30	2.95	2.21	1.07	.85	.32

UNEQUAL ANGLES—continued

See Notes.	Size, Inches.	Area in sq. in.	Wt. per ft. lb.	Radii, inches.		Distances to C. of G.		Moments of Inertia about		Section Moduli about		Radii of Gyration about			Tan α .
				Root R.	Toe r.	C.	D.	xx.	yy.	xx.	yy.	xx.	yy.	zz.	
*	$8 \times 3\frac{1}{2} \times$ (up to)	4.17	14.19	.47	.33	2.89	0.66	27.79	3.39	5.44	1.20	2.58	0.90	.73	.21
	$8 \times 4 \times$ (up to)	6.80	23.11	.48	.34	3.00	0.77	44.24	5.28	8.85	1.93	2.55	0.88	.72	.21
	$8 \times 4 \times$ (up to)	5.75	19.55	.48	.34	2.83	0.85	37.95	6.50	7.34	2.06	2.57	1.06	.85	.26
	$8 \times 4 \times$ (up to)	8.44	28.69	.48	.34	2.93	0.94	54.36	9.14	10.73	2.99	2.54	1.04	.84	.26
*	$8 \times 6 \times$ (up to)	6.75	22.95	.54	.38	2.44	1.45	43.47	21.08	7.82	4.63	2.54	1.77	1.29	.55
	$8 \times 6 \times$ (up to)	11.48	39.05	.54	.38	2.59	1.60	71.49	34.29	13.22	7.79	2.49	1.73	1.27	.54
	$9 \times 4 \times$ (up to)	6.25	21.25	.51	.36	3.27	0.80	52.46	6.65	9.16	2.08	2.90	1.03	.84	.22
	$9 \times 4 \times$ (up to)	10.61	36.07	.51	.36	3.43	0.95	86.13	10.60	15.47	3.47	2.85	1.00	.83	.21

TABLE 3
TEES

Size $\times$ thickness $B \times A \times t$	Area sq. in.	Radii		Wt. per ft.	Di- men- sion C	Moments of Inertia about		Section Moduli about		Radii of Gyration	
		Root R	Toe r			xx	yy	xx	yy	xx	yy
$1\frac{1}{2} \times 1\frac{1}{2} \times \frac{1}{8}$	0.69	.21	.15	2.36	0.46	0.14	0.07	0.13	0.09	0.44	0.31
$2 \times 2 \times \frac{1}{8}$	0.94	.24	.17	3.21	0.58	0.34	0.16	0.24	0.16	0.60	0.41
$2 \times 2 \times \frac{1}{4}$	1.16	"	"	3.94	0.60	0.41	0.20	0.29	0.20	0.59	0.42
$2 \times 2 \times \frac{3}{8}$	1.37	"	"	4.64	0.63	0.47	0.25	0.34	0.25	0.59	0.43
$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{8}$	1.20	.27	.19	4.07	0.70	0.68	0.30	0.38	0.24	0.75	0.50
$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$	1.47	"	"	5.00	0.72	0.82	0.39	0.46	0.31	0.75	0.51
$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{3}{8}$	1.74	"	"	5.92	0.75	0.96	0.47	0.55	0.38	0.74	0.52
$3 \times 3 \times \frac{1}{8}$	1.79	.30	.21	6.07	0.84	1.46	0.67	0.67	0.44	0.90	0.61
$3 \times 3 \times \frac{1}{4}$	2.12	"	"	7.20	0.87	1.71	0.81	0.80	0.54	0.90	0.62
$4 \times 3 \times \frac{1}{8}$	2.50	0.33	.23	8.49	0.77	1.86	1.91	0.83	0.96	0.86	0.87
$4 \times 3 \times \frac{1}{4}$	3.26	"	"	11.09	0.82	2.37	2.60	1.08	1.30	0.85	0.89
$4 \times 4 \times \frac{1}{8}$	2.87	.36	.25	9.77	1.10	4.19	1.90	1.45	0.95	1.21	0.81
$4 \times 4 \times \frac{1}{4}$	3.76	"	"	12.79	1.16	5.40	2.59	1.90	1.30	1.20	0.83
$5 \times 3 \times \frac{1}{8}$	2.88	.36	.25	9.79	0.69	1.97	3.72	0.85	1.49	0.83	1.14
$5 \times 3 \times \frac{1}{4}$	3.77	"	"	12.80	0.74	2.51	5.04	1.11	2.01	0.82	1.16
$5 \times 4 \times \frac{1}{8}$	3.25	.39	.27	11.06	1.0	4.47	3.70	1.49	1.48	1.17	1.07
$5 \times 4 \times \frac{1}{4}$	4.27	"	"	14.50	1.05	5.77	5.02	1.96	2.01	1.16	1.09
$6 \times 3 \times \frac{1}{8}$	3.26	.39	.27	11.08	0.63	2.06	6.40	0.87	2.13	0.80	1.40
$6 \times 3 \times \frac{1}{4}$	4.27	"	"	14.52	0.68	2.63	8.67	1.14	2.89	0.78	1.42
$6 \times 4 \times \frac{1}{8}$	4.77	.42	.29	16.22	0.97	6.07	8.64	2.00	2.88	1.13	1.35
$6 \times 4 \times \frac{1}{4}$	5.88	"	"	19.99	1.03	7.33	10.93	2.46	3.64	1.12	1.36
$6 \times 6 \times \frac{1}{8}$	5.77	.48	.34	19.62	1.63	19.04	8.56	4.36	2.85	1.83	1.22
$6 \times 6 \times \frac{1}{4}$	7.13	"	"	24.23	1.69	23.31	10.87	5.40	3.62	1.81	1.23

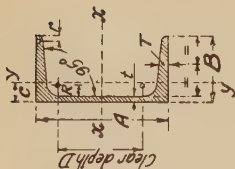
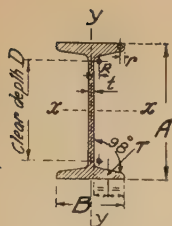


TABLE 4

CHANNELS

Each of the channels listed under can be rolled to a greater weight per ft. by raising the rolls and thus increasing the web thickness (dimensions A and T remain constant).

Size A × B inches.	Wt. per ft. lb.	Thickness, inches.		Radii, inches.		Area in sq. in.	Moments of Inertia about		Radii of Gyra- tion about		Sect. Moduli about		Distances C, D,	
		Web t.	Flange T.	Root R.	Toe r.		xx.	yy.	xx.	yy.	xx.	yy.		
3 × 1½	4.60	.20	.28	.30	.15	1.35	1.82	0.26	1.16	0.44	1.22	0.26	0.48	1.78
4 × 2	7.09	.24	.31	.36	.18	2.09	5.06	0.70	1.56	0.68	2.53	0.50	0.60	2.57
5 × 2½	10.22	.25	.38	.42	.21	3.01	11.87	1.64	1.99	0.74	4.75	0.95	0.77	3.28
6 × 3	12.41	.25	.38	.48	.24	3.65	21.27	2.83	2.41	0.88	7.09	1.34	0.89	4.12
6 × 3	16.51	.38	.48	.48	.24	4.86	26.38	3.70	2.33	0.87	8.76	1.77	0.91	3.93
6 × 3½	16.48	.28	.48	.54	.27	4.85	28.88	5.29	2.44	1.05	9.63	2.25	1.14	3.77
7 × 3	14.22	.26	.42	.48	.24	4.18	32.75	3.26	2.80	0.88	9.36	1.53	0.88	5.04
7 × 3½	18.28	.30	.50	.54	.27	5.38	42.83	5.83	2.82	1.04	12.24	2.42	1.09	4.73
8 × 3	15.96	.28	.44	.48	.24	4.69	46.72	3.58	3.16	0.87	11.68	1.65	0.83	6.01
8 × 3½	20.21	.32	.52	.54	.27	5.94	60.57	6.37	3.19	1.04	15.14	2.60	1.05	5.70
9 × 3	17.46	.30	.44	.48	.24	5.14	62.52	3.75	3.49	0.86	13.89	1.69	0.78	7.01
9 × 3½	22.27	.34	.54	.54	.27	6.55	82.62	6.90	3.55	1.03	18.36	2.76	1.00	6.66
10 × 3	19.28	.32	.45	.48	.24	5.67	82.66	3.98	3.82	0.84	16.53	1.76	0.74	7.99
10 × 3½	24.46	.36	.56	.54	.27	7.19	109.52	7.42	3.90	1.02	21.90	2.93	0.97	7.62
11 × 3	26.78	.38	.58	.54	.27	7.88	141.87	7.93	4.24	1.00	25.80	3.09	0.93	8.58
11 × 3½	26.37	.38	.50	.54	.27	7.76	159.73	7.15	4.54	0.96	26.62	2.68	0.83	9.74
12 × 4	31.33	.40	.60	.60	.30	9.21	200.09	12.12	4.66	1.15	33.35	4.12	1.06	9.39
13 × 4	33.18	.40	.62	.60	.30	9.76	246.86	12.76	5.03	1.14	37.98	4.31	1.04	10.35
15 × 4	39.37	.41	.62	.60	.30	10.70	349.10	13.34	5.71	1.12	46.55	4.40	0.97	12.35
17 × 4	44.34	.43	.68	.60	.30	13.04	520.18	15.26	6.32	1.08	61.10	4.96	0.92	14.24



BEAMS

Size A × B inches.	Weight per ft. in lbs.	Thickness, inches.		Radii, inches.		Area in sq. in.	Moments of Inertia about		Section Moduli		Radii of Gyratation about		Dis- tance D
		Web <i>t</i> .	Flange T.	Root R.	Toe r.		<i>xx</i>	<i>yy</i>	<i>Zxx</i>	<i>Zyy</i>	<i>xx</i>	<i>yy</i>	
3 × 1½	4	-16	-249	-25	-12	1-18	1-66	0-13	1-11	0-17	1-19	0-33	1-98
3 × 3	8-5	-20	-332	-37	-18	2-52	3-81	1-25	2-54	0-83	1-23	0-70	1-50
4 × 1½	5	-17	-239	-27	-13	1-47	3-66	0-19	1-83	0-21	1-58	0-36	2-95
4 × 3	10	-24	-347	-37	-18	2-94	7-79	1-33	3-89	0-88	1-63	0-67	2-47
4½ × 1½	6-5	-18	-325	-27	-13	1-91	6-73	0-26	2-83	0-30	1-88	0-37	3-52
5 × 3	11	-22	-376	-37	-18	3-26	13-68	1-45	5-47	0-97	2-05	0-67	3-40
5 × 4½	20	-29	-513	-49	-24	5-88	25-03	6-59	10-01	2-93	2-06	1-06	2-84
6 × 3	12	-23	-377	-37	-18	3-53	20-99	1-46	7-00	0-97	2-44	0-64	4-42
6 × 4½	20	-37	-431	-49	-24	5-89	34-71	5-40	11-57	2-40	2-43	0-96	4-00
6 × 5	25	-41	-52	-53	-26	7-37	43-69	9-10	14-56	3-64	2-44	1-11	3-72
7 × 4	16	-25	-387	-45	-22	4-75	39-51	3-37	11-29	1-69	2-89	0-84	5-18
8 × 4	18	-28	-398	-45	-22	5-30	55-63	3-51	13-91	1-75	3-24	0-81	6-17
8 × 5	28	-35	-575	-53	-26	8-28	89-69	10-19	22-42	4-08	3-29	1-11	5-60
8 × 6	35	-35	-648	-61	-30	10-30	115-06	19-54	28-76	6-51	3-34	1-38	5-26
9 × 4	21	-30	-457	-45	-22	6-18	81-13	4-15	18-03	2-07	3-62	0-82	7-05
9 × 7	50	-40	-825	-69	-34	14-71	208-13	40-17	46-25	11-48	3-76	1-65	5-70
10 × 4½	25	-30	-505	-49	-24	7-35	122-34	6-49	24-47	2-88	4-08	0-94	7-85
10 × 5	30	-36	-552	-53	-26	8-85	146-23	9-73	29-25	3-89	4-06	1-05	7-64
10 × 6	40	-36	-709	-61	-30	11-77	204-80	21-76	40-96	7-25	4-17	1-36	7-14
10 × 8	55	-40	-783	-77	-38	16-18	288-69	54-74	57-74	13-69	4-22	1-84	6-57
12 × 5	32	-35	-550	-53	-26	9-45	221-07	9-69	36-84	3-88	4-84	1-01	9-66
12 × 6	44	-40	-717	-61	-30	13-00	316-76	22-12	52-79	7-37	4-94	1-30	9-12
12 × 6	54	-50	-883	-61	-30	15-89	375-77	28-28	62-63	9-43	4-86	1-33	8-78
12 × 8	65	-43	-904	-77	-38	19-12	487-77	65-18	81-30	16-30	5-05	1-85	8-33
13 × 5	35	-35	-604	-53	-26	10-30	283-51	10-82	43-62	4-33	5-25	1-03	10-55
14 × 6	46	-40	-698	-61	-30	13-59	442-57	21-45	63-22	7-15	5-71	1-26	11-14
14 × 6	57	-50	-873	-61	-30	16-78	533-34	27-94	76-19	9-31	5-64	1-29	10-80
14 × 8	70	-46	-920	-77	-38	20-59	705-58	66-67	100-80	16-67	5-85	1-80	10-30
15 × 5	42	-42	-647	-53	-26	12-36	428-49	11-81	57-13	4-72	5-89	0-98	12-46
15 × 6	45	-38	-655	-61	-30	13-24	491-91	19-87	65-59	6-62	6-10	1-23	12-24
16 × 6	50	-40	-726	-61	-30	14-71	618-09	22-47	77-26	7-49	6-48	1-24	13-10
16 × 6	62	-55	-847	-61	-30	18-21	725-05	27-14	90-63	9-05	6-31	1-22	12-86
16 × 8	75	-48	-938	-77	-38	22-06	973-91	68-30	121-74	17-08	6-64	1-76	12-27
18 × 6	55	-42	-757	-61	-30	16-18	841-76	23-64	93-53	7-88	7-21	1-21	15-04
18 × 7	75	-55	-928	-69	-34	22-09	1151-18	46-56	127-91	13-30	7-22	1-45	14-50
18 × 8	80	-50	-950	-77	-38	23-53	1292-07	69-43	143-56	17-36	7-41	1-72	14-24
20 × 6½	65	-45	-820	-65	-32	19-12	1226-17	32-56	122-62	10-02	8-01	1-31	16-82
20 × 7½	89	-60	1-010	-73	-36	26-19	1672-85	62-54	167-29	16-68	7-99	1-55	16-22
22 × 7	75	-50	-834	-69	-34	22-06	1676-80	41-07	152-44	11-63	8-72	1-36	16-69
24 × 7½	95	-57	1-011	-73	-36	27-94	2533-04	62-54	211-09	16-68	9-52	1-50	20-22

TABLE 6
MAXIMAL SIZES TO WHICH PLATES ARE ROLLED

Thickness inches.	Greatest			Thickness inches.	Greatest		
	Length feet.	Breadth feet.	Area sq. ft.		Length feet.	Breadth feet.	Area sq. ft.
$\frac{1}{4}$	35	7.5	250	$\frac{5}{8}$	65	10.0	350
$\frac{5}{16}$	40	8.0	260	$\frac{11}{16}$	"	10.5	"
$\frac{3}{8}$	65	8.5	330	$\frac{3}{4}$	"	11.0	"
$\frac{7}{16}$	"	9.0	350	$\frac{7}{8}$	"	11.5	"
$\frac{1}{2}$	"	9.5	"	1	"	"	"
$\frac{9}{16}$	"	"	"	$1\frac{1}{8}$	"	"	300

Some mills can roll exceptionally long plates and others exceptionally broad plates, but in neither case can the greatest plate area be exceeded. Hence the obtainable length of a maximum breadth plate will be the max. plate area $\div$ max. breadth; e.g., the obtainable length of a $\frac{5}{16}$ -in. thick plate of the maximum breadth of 8 ft. is 260 sq. ft. $\div$ 8 ft. = 32.5 ft.

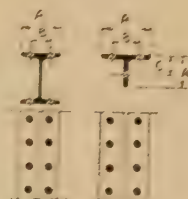
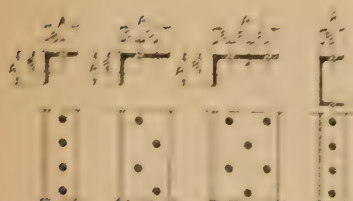
TABLE 7
MAXIMAL SIZES TO WHICH FLAT BARS ARE
ROLLED

Width in inches.	Thickness in inches.		Width in inches.	Thickness in inches.	
	Minimum.	Maximum.		Minimum.	Maximum.
$\frac{3}{4}$	$\frac{1}{8}$	$\frac{5}{8}$	$4\frac{1}{4}$ rising by $\frac{1}{4}$ to 6	$\frac{3}{16}$	$1\frac{1}{2}$
$\frac{7}{8}$	$\frac{1}{8}$	$\frac{3}{4}$	$6\frac{1}{4}$ rising by $\frac{1}{4}$ to 7	$\frac{3}{16}$	2
1	$\frac{1}{8}$	$\frac{7}{8}$	7 rising by 1 to 13	$\frac{3}{8}$	2
$1\frac{1}{8}$ rising by $\frac{1}{8}$ to 2	$\frac{1}{8}$	1	14, 16 and 18 .	$\frac{3}{8}$	$2\frac{1}{2}$
$2\frac{1}{8}$ rising by $\frac{1}{8}$ to 4	$\frac{5}{32}$	$1\frac{1}{2}$	$20\frac{1}{4}$	$\frac{5}{16}$	$2\frac{5}{8}$

Note.—The thicker the flat the greater is the tendency towards rounded edges.

TABLE 8

THE USUAL HOLING FOR ANGLES, CHANNELS,
JOISTS AND TEES



All Dimensions in inches.						Max. Dia. of Hole.
A.	B.	C.	D.	E.	F.	
9	—	3	4	3	$2\frac{1}{2}$	1 C & D
8	—	3	3	$2\frac{1}{4}$	$2\frac{1}{2}$	1 "
7	—	$2\frac{1}{2}$	3	$2\frac{1}{4}$	$1\frac{3}{4}$	1 "
$6\frac{1}{2}$	—	$2\frac{1}{2}$	$2\frac{1}{2}$	$2\frac{1}{4}$	$1\frac{1}{2}$	1 "
6	$3\frac{1}{2}$	$2\frac{1}{4}$	2		1	"
6	$3\frac{1}{4}$	$2\frac{1}{4}$	$2\frac{1}{4}$		1	"
$5\frac{1}{2}$	3	$2\frac{1}{4}$	2	C D =	B =	1
5	$2\frac{3}{4}$	2	$1\frac{3}{4}$		D =	$\frac{13}{16}$
$4\frac{1}{2}$	$2\frac{1}{2}$	2	$1\frac{1}{2}$			$\frac{13}{16}$
4	$2\frac{1}{4}$	In very thick angles dimensions B, C and E may be increased or, alternatively, the diameter of hole decreased, in order to provide clearance for rivet head or nut.				1
$3\frac{1}{2}$	2					$\frac{7}{8}$
3	$1\frac{3}{4}$					$\frac{7}{8}$
$2\frac{3}{4}$	$1\frac{5}{8}$					$\frac{3}{4}$
$2\frac{1}{2}$	$1\frac{3}{8}$					$\frac{3}{4}$
$2\frac{1}{4}$	$1\frac{1}{2}$					$\frac{3}{4}$
2	$1\frac{1}{8}$					$\frac{3}{8}$
$1\frac{3}{4}$	1					$\frac{3}{8}$
$1\frac{1}{2}$	$\frac{7}{8}$					$\frac{3}{8}$

All Dimensions in inches.			C.	Max. Dia. of Hole.
A.	B.	Max. Dia. of Hole.		
8	$4\frac{1}{2}$	1	—	—
$7\frac{1}{2}$	$4\frac{1}{2}$	1	—	—
7	4	$\frac{7}{8}$	—	—
$6\frac{1}{2}$	$3\frac{3}{4}$	$\frac{7}{8}$	—	—
6	$3\frac{1}{2}$	$\frac{11}{16}$	$3\frac{1}{2}$	1
$5\frac{1}{2}$	$3\frac{1}{4}$	$\frac{7}{8}$	3	1
5	$2\frac{3}{4}$	$\frac{3}{4}$	$2\frac{1}{2}$	1
$4\frac{1}{2}$	$2\frac{1}{2}$	$\frac{3}{4}$	$2\frac{1}{2}$	$\frac{7}{8}$
4	$2\frac{1}{4}$	$\frac{3}{4}$	$2\frac{1}{2}$	$\frac{7}{8}$
$3\frac{1}{2}$	$1\frac{3}{4}$	$\frac{9}{16}$	2	$\frac{7}{8}$
3	$1\frac{1}{2}$	$\frac{1}{2}$	$1\frac{3}{4}$	$\frac{11}{16}$
$2\frac{1}{2}$	$1\frac{1}{2}$	$\frac{1}{2}$	$1\frac{3}{4}$	$\frac{11}{16}$
$2\frac{1}{4}$	$1\frac{1}{2}$	$\frac{1}{2}$	$1\frac{3}{4}$	$\frac{11}{16}$
$2\frac{1}{2}$	$1\frac{1}{2}$	$\frac{1}{2}$	$1\frac{3}{4}$	$\frac{11}{16}$
2	1	$\frac{3}{8}$	1	$\frac{11}{16}$
$1\frac{3}{4}$	$\frac{3}{4}$	$\frac{1}{4}$	$\frac{7}{8}$	$\frac{3}{8}$
$1\frac{1}{2}$	$\frac{3}{4}$	$\frac{1}{4}$	$\frac{7}{8}$	$\frac{3}{8}$

See Chapter IV. for mnemonic for the rivet lines of angles, channel flanges and stems of tees.

TABLE 9

AREAS TO BE DEDUCTED FOR EACH RIVET
HOLE IN TENSION MEMBERS

Thickness of Metal.		Diameter of Rivet or Bolt Hole.								
		$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1
$\frac{1}{4}$	$\frac{1}{16}$	.03	.04	.04	.04	.05	.05	.06	.06	.06
	$\frac{3}{16}$	.06	.07	.08	.09	.09	.10	.11	.12	.13
	$\frac{1}{8}$	.09	.11	.12	.13	.14	.15	.16	.18	.19
	$\frac{1}{4}$	.13	.14	.16	.17	.19	.20	.22	.23	.25
$\frac{1}{2}$	$\frac{5}{16}$	.16	.18	.20	.22	.23	.25	.27	.29	.31
	$\frac{3}{8}$	.19	.21	.23	.26	.28	.31	.33	.35	.38
	$\frac{7}{16}$	.22	.25	.27	.30	.33	.36	.38	.41	.44
	$\frac{1}{2}$	.25	.28	.31	.34	.38	.41	.44	.47	.50
$\frac{3}{4}$	$\frac{9}{16}$	.28	.32	.35	.39	.42	.46	.49	.53	.56
	$\frac{5}{8}$	.31	.35	.39	.43	.47	.51	.55	.59	.63
	$\frac{11}{16}$	.34	.39	.43	.47	.52	.56	.60	.65	.69
	$\frac{3}{4}$	.38	.42	.47	.52	.56	.61	.66	.70	.75
1	$\frac{13}{16}$	.41	.46	.51	.56	.61	.66	.71	.76	.81
	$\frac{7}{8}$	.44	.49	.55	.60	.66	.71	.76	.82	.88
	$\frac{15}{16}$	.47	.53	.59	.65	.70	.76	.82	.88	.94
	1	.50	.56	.63	.69	.75	.81	.88	.94	1.00

TABLE 10

AREAS TO BE DEDUCTED FROM ANGLES



Where a single angle is riveted through only one leg then the effective net area shall be obtained by deducting from the gross area the hole in the riveted leg and also half of the gross area of the unriveted leg.

Angle Leg.	$\frac{1}{2}$ Leg Area of Thickness.					$\frac{1}{2}$ Leg Area + $\frac{3}{4}$ " dia. Hole.					$\frac{1}{2}$ Leg Area + $\frac{7}{8}$ " dia. Hole.				
	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$
2	.22	.26	—	—	—	.41	.50	—	—	—	—	—	—	—	—
$2\frac{1}{2}$	.28	.34	.40	.45	.50	.47	.57	.68	.78	.88	—	—	—	—	—
3	.34	.42	.49	.56	.63	.53	.65	.77	.89	1.00	.56	.69	.82	.94	1.06
$3\frac{1}{2}$	.41	.50	.59	.67	.75	.59	.73	.87	1.00	1.13	.63	.77	.91	1.05	1.19
4	.47	.58	.68	.78	.88	.66	.81	.96	1.11	1.25	.69	.85	1.01	1.16	1.31
$4\frac{1}{2}$	.53	.65	.77	.89	1.00	.71	.89	1.05	1.22	1.38	.74	.93	1.10	1.27	1.44

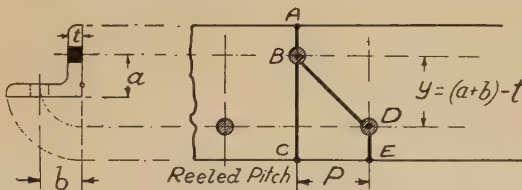
TABLE 11

NET SECTIONAL AREAS OF ANGLES WITH SINGLE RIVET LINES

When the reeled pitch P has the tabulated values the tendency to tear under tension (in terms of the *British Standard Specification for Girder Bridges*, Part 4, paragraph 4, is the same along the perpendicular line ABC as along the zig-zag line $ABDE$.

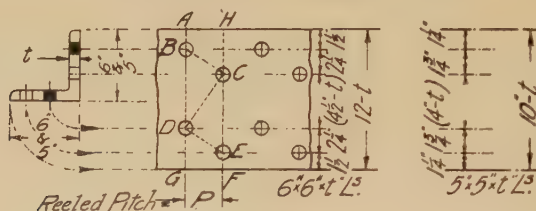
If the reeled pitch used be greater than the tabulated value, the tear is ABC .

If the reeled pitch used be less than the tabulated value, the tear is $ABDE$.



Distance y in inches.	Reeled Pitch P for a Diameter of Hole d of				
	$\frac{3}{4}$ in.	$1\frac{1}{8}$ in.	$\frac{7}{8}$ in.	$1\frac{5}{8}$ in.	1 in.
2	2.56	2.64	2.72	2.80	2.87
$2\frac{1}{8}$	2.66	2.74	2.82	2.90	2.98
$2\frac{1}{4}$	2.76	2.84	2.92	3.00	3.08
$2\frac{3}{8}$	2.86	2.94	3.02	3.10	3.18
$2\frac{1}{2}$	2.96	3.04	3.12	3.20	3.28
$2\frac{5}{8}$	3.06	3.14	3.22	3.30	3.38
$2\frac{3}{4}$	3.16	3.24	3.32	3.40	3.48
$2\frac{7}{8}$	3.26	3.34	3.42	3.50	3.58
3	3.35	3.44	3.52	3.60	3.68
$3\frac{1}{8}$	3.45	3.54	3.62	3.70	3.78
$3\frac{1}{4}$	3.55	3.63	3.72	3.80	3.88
$3\frac{3}{8}$	3.65	3.73	3.82	3.90	3.98
$3\frac{1}{2}$	3.74	3.83	3.91	4.00	4.08
$3\frac{3}{4}$	3.94	4.02	4.11	4.19	4.28
4	4.13	4.22	4.30	4.39	4.47
$4\frac{1}{4}$	4.32	4.41	4.50	4.58	4.67
$4\frac{1}{2}$	4.52	4.60	4.69	4.78	4.86
$4\frac{3}{4}$	4.71	4.80	4.88	4.97	5.06
5	4.90	4.99	5.08	5.16	5.25
$5\frac{1}{2}$	5.28	5.37	5.46	5.55	5.64

TABLE 12
NET SECTIONAL AREAS OF ANGLES WITH DOUBLE
RIVET LINES



In terms of the B.S.S. Girder Bridges (4/4) the net sectional area is the least net area obtained by considering any zig-zag or transverse tear through adjacent rivet holes.

Transverse net area = {length ABDG minus 2 holes} $\times$ thickness.

Zig-zag tear, e.g., path ABCDEF,

net area = $\{(AB - \frac{1}{2} \text{ hole}) + \frac{4}{5}[(BC - 1 \text{ hole}) + (CD - 1 \text{ hole}) + (DE - 1 \text{ hole})] + (EF - \frac{1}{2} \text{ hole})\} \times \text{thickness.}$

The minimum net area, however, cannot be less than that obtained by assuming all four holes in one transverse section AG.

$P = 1\frac{1}{2}"$. Path ABCDEF.					
Dia. of hole = $\frac{3}{4}"$	$\frac{1}{8}"$	$\frac{7}{8}"$	$\frac{1}{2}"$	$\frac{1}{4}"$	1"
$5" \times 5" \times \frac{3}{8}"$	2.54	2.46	2.38	2.30	2.22
$\frac{1}{2}"$	3.34	3.24	3.13	3.03	2.92
$\frac{5}{8}"$	4.12	3.99	3.86	3.73	3.59
$\frac{3}{4}"$	4.88	4.72	4.56	4.40	4.24
$\frac{7}{8}"$	5.61	5.42	5.24	5.05	4.87
1"	6.32	6.11	5.90	5.68	5.47

$P = 1\frac{3}{4}"$. Path ABCDEF.					
Dia. of hole = $\frac{3}{4}"$	$\frac{1}{8}"$	$\frac{7}{8}"$	$\frac{1}{2}"$	$\frac{1}{4}"$	1"
$5" \times 5" \times \frac{3}{8}"$	2.67	2.59	2.51	2.43	2.35
$\frac{1}{2}"$	3.52	3.41	3.31	3.20	3.09
$\frac{5}{8}"$	4.34	4.21	4.08	3.94	3.81
$\frac{3}{4}"$	5.15	4.99	4.83	4.67	4.51
$\frac{7}{8}"$	5.93	5.74	5.55	5.37	5.18
1"	6.68	6.47	6.26	6.05	5.83

$P = 2"$. Path ABCDEF.					
Dia. of hole = $\frac{3}{4}"$	$\frac{1}{8}"$	$\frac{7}{8}"$	$\frac{1}{2}"$	$\frac{1}{4}"$	1"
$5" \times 5" \times \frac{3}{8}"$	2.82	2.74	2.66	2.58	2.50
$\frac{1}{2}"$	3.71	3.61	3.50	3.40	3.29
$\frac{5}{8}"$	4.59	4.46	4.32	4.19	4.06
$\frac{3}{4}"$	5.44	5.28	5.12	4.96	4.80
$\frac{7}{8}"$	6.27	6.09	5.90	5.72	5.53
1"	7.09	6.87	6.66	6.45	6.24

$P = 2\frac{1}{4}"$. Path ABCDEF.					
Dia. of hole = $\frac{3}{4}"$	$\frac{1}{8}"$	$\frac{7}{8}"$	$\frac{1}{2}"$	$\frac{1}{4}"$	1"
$5" \times 5" \times \frac{3}{8}"$	2.97	2.89	2.81	2.73	2.65
$\frac{1}{2}"$	3.92	3.81	3.71	3.60	3.50
$\frac{5}{8}"$	4.85	4.72	4.58	4.45	4.32
$\frac{3}{4}"$	5.76	5.60	5.44	5.28	5.11
$\frac{7}{8}"$	6.61	6.46	6.27	6.07	5.90
1"	7.43	7.26	7.08	6.87	6.66
Path ABCE					

$P = 2\frac{1}{2}"$. Path ABDG.					
Dia. of hole = $\frac{3}{4}"$	$\frac{1}{8}"$	$\frac{7}{8}"$	$\frac{1}{2}"$	$\frac{1}{4}"$	1"
$5" \times 5" \times \frac{3}{8}"$	3.05	3.00	2.95	2.88	2.82
$\frac{1}{2}"$	4.00	3.94	3.87	3.78	3.70
$\frac{5}{8}"$	4.92	4.84	4.76	4.65	4.54
$\frac{3}{4}"$	5.81	5.72	5.62	5.49	5.36
$\frac{7}{8}"$	6.67	6.56	6.45	6.29	6.14
1"	7.50	7.38	7.24	7.07	6.89

$P = 2\frac{3}{4}"$, 3", 4", etc. All have path ABDG.					
Dia. of hole = $\frac{3}{4}"$	$\frac{1}{8}"$	$\frac{7}{8}"$	$\frac{1}{2}"$	$\frac{1}{4}"$	1"
$5" \times 5" \times \frac{3}{8}"$	3.05	3.00	2.95	2.91	2.86
$\frac{1}{2}"$	4.00	3.94	3.88	3.81	3.75
$\frac{5}{8}"$	4.92	4.84	4.77	4.69	4.61
$\frac{3}{4}"$	5.81	5.72	5.63	5.53	5.44
$\frac{7}{8}"$	6.67	6.56	6.45	6.34	6.23
1"	7.50	7.38	7.25	7.13	7.00

TABLE 12—continued
See previous page for diagram.

$P = 1\frac{1}{2}"$. Path $ABCDEF$.					
Dia. of hole = $\frac{3}{4}"$	$\frac{1}{16}"$	$\frac{7}{8}"$	$\frac{1}{16}"$	1"	
$6" \times 6" \times \frac{3}{8}"$	3.11	3.03	2.95	2.87	2.79
* $\frac{1}{2}"$	4.10	3.99	3.89	3.78	3.68
$\frac{5}{8}"$	5.06	4.93	4.80	4.66	4.51
$\frac{3}{4}"$	6.01	5.85	5.69	5.53	5.37
$\frac{7}{8}"$	6.93	6.74	6.55	6.37	6.18
1"	7.82	7.61	7.39	7.18	6.97

$P = 1\frac{3}{4}"$. Path $ABCDEF$.					
Dia. of hole = $\frac{3}{4}"$	$\frac{1}{16}"$	$\frac{7}{8}"$	$\frac{1}{16}"$	1"	
$6" \times 6" \times \frac{3}{8}"$	3.22	3.14	3.06	2.98	2.91
$\frac{1}{2}"$	4.25	4.15	4.04	3.93	3.83
$\frac{5}{8}"$	5.26	5.13	4.99	4.86	4.73
$\frac{3}{4}"$	6.24	6.08	5.92	5.76	5.61
$\frac{7}{8}"$	7.20	7.02	6.83	6.64	6.45
1"	8.14	7.93	7.72	7.51	7.29

$P = 2"$. Path $ABCDEF$.					
Dia. of hole = $\frac{3}{4}"$	$\frac{1}{16}"$	$\frac{7}{8}"$	$\frac{1}{16}"$	1"	
$6" \times 6" \times \frac{3}{8}"$	3.35	3.27	3.19	3.11	3.03
$\frac{1}{2}"$	4.42	4.32	4.21	4.10	4.00
$\frac{5}{8}"$	5.47	5.34	5.21	5.08	4.94
$\frac{3}{4}"$	6.50	6.35	6.19	6.03	5.86
$\frac{7}{8}"$	7.51	7.32	7.13	6.96	6.76
1"	8.49	8.28	8.07	7.87	7.64

$P = 2\frac{1}{4}"$. Path $ABCDEF$.					
Dia. of hole = $\frac{3}{4}"$	$\frac{1}{16}"$	$\frac{7}{8}"$	$\frac{1}{16}"$	1"	
$6" \times 6" \times \frac{3}{8}"$	3.49	3.41	3.33	3.25	3.17
$\frac{1}{2}"$	4.60	4.50	4.39	4.29	4.18
$\frac{5}{8}"$	5.70	5.57	5.44	5.30	5.17
$\frac{3}{4}"$	6.78	6.62	6.46	6.30	6.14
$\frac{7}{8}"$	7.83	7.65	7.46	7.28	7.09
1"	8.87	8.66	8.44	8.23	8.02

$P = 2\frac{1}{2}"$. Path $ABCDEF$.					
Dia. of hole = $\frac{3}{4}"$	$\frac{1}{16}"$	$\frac{7}{8}"$	$\frac{1}{16}"$	1"	
$6" \times 6" \times \frac{3}{8}"$	3.64	3.56	3.48	3.40	3.32
$\frac{1}{2}"$	4.81	4.70	4.59	4.49	4.38
$\frac{5}{8}"$	5.96	5.82	5.69	5.56	5.43
$\frac{3}{4}"$	7.08	6.93	6.77	6.61	6.45
$\frac{7}{8}"$	8.19	8.01	7.82	7.63	7.45
1"	9.28	9.07	8.86	8.64	8.43

$P = 2\frac{3}{4}"$. Path $ABCDEF$.					
Dia. of hole = $\frac{3}{4}"$	$\frac{1}{16}"$	$\frac{7}{8}"$	$\frac{1}{16}"$	1"	
$6" \times 6" \times \frac{3}{8}"$	3.79	3.71	3.63	3.55	3.47
$\frac{1}{2}"$	5.00	4.90	4.80	4.69	4.58
$\frac{5}{8}"$	6.16	6.06	5.94	5.81	5.68
$\frac{3}{4}"$	7.31	7.18	7.05	6.91	6.75
$\frac{7}{8}"$	8.42	8.14	8.11	7.96	7.80
1"	9.49	9.32	9.14	8.97	8.79
Path $ABCEF$.					

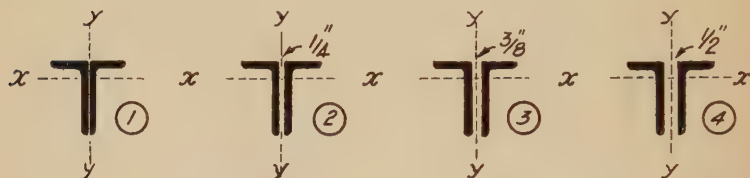
$P = 3"$. Path $ABGD$. $\leftarrow \rightarrow$					
Dia. of hole = $\frac{3}{4}"$	$\frac{1}{16}"$	$\frac{7}{8}"$	$\frac{1}{16}"$	1"	
$6" \times 6" \times \frac{3}{8}"$	3.80	3.75	3.70	3.66	3.59
$\frac{1}{2}"$	5.00	4.94	4.88	4.81	4.73
$\frac{5}{8}"$	6.17	6.09	6.02	5.94	5.83
$\frac{3}{4}"$	7.31	7.22	7.12	7.03	6.90
$\frac{7}{8}"$	8.42	8.31	8.20	8.09	7.94
1"	9.50	9.38	9.25	9.13	8.95
Path $ABCEF$.					$\leftarrow \rightarrow$

$P = 3\frac{1}{4}"$, $3\frac{1}{2}"$, $3\frac{3}{4}"$, 4", etc. All have path $ABDQ$.					
Dia. of hole = $\frac{3}{4}"$	$\frac{1}{16}"$	$\frac{7}{8}"$	$\frac{1}{16}"$	1"	
$6" \times 6" \times \frac{3}{8}"$	3.80	3.75	3.70	3.66	3.61
$\frac{1}{2}"$	5.00	4.94	4.88	4.81	4.75
$\frac{5}{8}"$	6.17	6.09	6.02	5.94	5.86
$\frac{3}{4}"$	7.31	7.22	7.12	7.03	6.94
$\frac{7}{8}"$	8.42	8.31	8.20	8.09	7.98
1"	9.50	9.38	9.25	9.13	9.00

* The table of $P = 1\frac{1}{2}"$ for $6" \times 6"$ angles gives net areas less than the specified minimum. Hence instead of using the table values the net areas are obtained on multiplying the angle thicknesses by (length $AG - 4$ holes).

TABLE 13

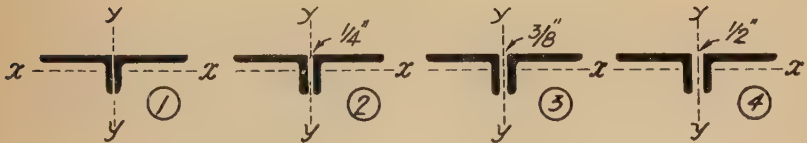
RADII OF GYRATION OF TWO UNEQUAL ANGLES
PLACED BACK TO BACK, WITH THE SHORT
LEGS TURNED OUT



Size of two Angles, in inches.	Area of two Angles, in sq. inches.	Axis xx , in inches.	Axis yy , in inches.			
			(1)	(2)	(3)	(4)
$3 \times 2\frac{1}{2} \times \frac{1}{16}$	2.63	0.93	0.98	1.07	1.12	1.16
	3.24	0.93	0.99	1.08	1.13	1.18
	3.85	0.92	1.01	1.10	1.15	1.20
$3\frac{1}{2} \times 3 \times \frac{1}{16}$	3.13	1.09	1.18	1.27	1.31	1.36
	3.87	1.08	1.19	1.28	1.32	1.37
	4.60	1.08	1.21	1.30	1.34	1.39
$4 \times 3 \times \frac{5}{16}$	4.18	1.26	1.15	1.24	1.28	1.33
	4.97	1.25	1.16	1.25	1.29	1.34
	6.50	1.24	1.18	1.27	1.32	1.37
$4 \times 3\frac{1}{2} \times \frac{5}{16}$	5.35	1.24	1.40	1.49	1.53	1.58
	7.00	1.22	1.43	1.52	1.56	1.61
	8.60	1.21	1.46	1.55	1.59	1.64
$5 \times 3 \times \frac{5}{16}$	5.72	1.59	1.08	1.16	1.21	1.25
	7.50	1.58	1.10	1.19	1.24	1.29
	9.22	1.56	1.13	1.22	1.27	1.32
$5 \times 3\frac{1}{2} \times \frac{5}{16}$	6.10	1.58	1.32	1.40	1.45	1.49
	8.00	1.57	1.34	1.43	1.47	1.52
	9.85	1.55	1.36	1.45	1.50	1.54
$6 \times 3 \times \frac{5}{16}$	6.47	1.92	1.02	1.10	1.14	1.19
	8.50	1.91	1.04	1.13	1.17	1.22
	10.47	1.89	1.06	1.15	1.20	1.25
$6 \times 3\frac{1}{2} \times \frac{5}{16}$	6.84	1.92	1.24	1.32	1.36	1.41
	9.00	1.91	1.26	1.35	1.39	1.43
	11.09	1.89	1.29	1.37	1.42	1.47
$6 \times 4 \times \frac{5}{16}$	7.22	1.91	1.47	1.55	1.59	1.64
	9.50	1.90	1.49	1.58	1.62	1.67
	11.72	1.88	1.52	1.60	1.65	1.69

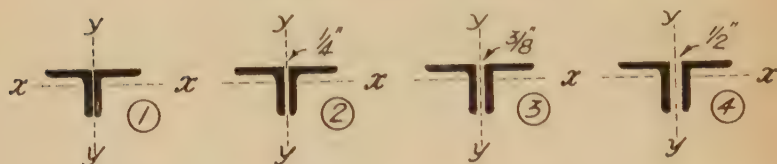
TABLE 14

RADII OF GYRATION OF TWO UNEQUAL ANGLES
PLACED BACK TO BACK, WITH LONG LEGS
TURNED OUT



Size of Angles in inches.	Area of two Angles, in sq. inches.	Axis xx , in inches.	Axis yy , in inches.			
			(1)	(2)	(3)	(4)
$3 \times 2\frac{1}{2} \times \frac{1}{4}$ 9.30-1.25-2.50	2.63	0.74	1.29	1.38	1.42	1.47
	3.24	0.73	1.30	1.40	1.44	1.49
	3.85	0.73	1.31	1.41	1.45	1.50
$3\frac{1}{2} \times 3 \times \frac{1}{4}$ 10.10-1.50-3.00	3.13	0.90	1.49	1.57	1.62	1.67
	3.87	0.89	1.50	1.59	1.63	1.69
	4.60	0.89	1.52	1.61	1.66	1.70
$4 \times 3 \times \frac{5}{16}$ 13.50-1.75-3.50	4.18	0.87	1.77	1.85	1.90	1.95
	4.98	0.87	1.78	1.88	1.92	1.97
	6.50	0.85	1.81	1.90	1.95	2.00
$4 \times 3\frac{1}{2} \times \frac{1}{2}$ 15.00-2.00-4.00	5.35	1.04	1.72	1.81	1.85	1.90
	7.00	1.03	1.74	1.85	1.88	1.93
	8.60	1.02	1.77	1.86	1.91	1.96
$5 \times 3 \times \frac{1}{2}$ 16.50-2.25-4.50	5.72	0.83	2.31	2.41	2.45	2.50
	7.50	0.82	2.34	2.44	2.48	2.53
	9.22	0.81	2.37	2.46	2.51	2.56
$5 \times 3\frac{1}{2} \times \frac{1}{2}$ 18.00-2.50-5.00	6.10	1.01	2.24	2.30	2.38	2.43
	8.00	1.00	2.27	2.36	2.40	2.46
	9.85	0.98	2.30	2.39	2.44	2.49
$6 \times 3 \times \frac{1}{2}$ 20.00-2.75-5.50	6.47	0.80	2.86	2.96	3.01	3.05
	8.50	0.78	2.88	2.99	3.04	3.08
	10.47	0.77	2.92	3.01	3.06	3.11
$6 \times 3\frac{1}{2} \times \frac{1}{2}$ 22.00-3.00-6.00	6.84	0.97	2.78	2.87	2.92	2.97
	9.00	0.96	2.81	2.90	2.95	3.00
	11.09	0.95	2.84	2.93	2.98	3.03
$6 \times 4 \times \frac{1}{2}$ 24.00-3.25-6.50	7.22	1.14	2.71	2.79	2.83	2.89
	9.50	1.13	2.74	2.83	2.87	2.91
	11.72	1.12	2.76	2.85	2.89	2.95

TABLE 15
RADI OF GYRATION OF TWO EQUAL ANGLES
PLACED BACK TO BACK



Size of Angles in inches.	Area of two Angles, in sq. inches.	Axis xx , in inches.	Axis yy , in inches.			
			(1)	(2)	(3)	(4)
$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$ $\frac{5}{16}$ $\frac{3}{8}$	2.38	0.76	1.03	1.12	1.17	1.21
	2.93	0.75	1.05	1.14	1.18	1.23
	3.47	0.74	1.06	1.15	1.20	1.25
$3 \times 3 \times \frac{1}{4}$ $\frac{5}{16}$ $\frac{3}{8}$	2.87	0.92	1.23	1.32	1.37	1.41
	3.56	0.91	1.24	1.33	1.38	1.43
	4.22	0.90	1.26	1.35	1.40	1.45
$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{5}{16}$ $\frac{3}{8}$ $\frac{1}{2}$	4.18	1.07	1.44	1.53	1.58	1.62
	4.97	1.06	1.46	1.55	1.59	1.64
	6.50	1.05	1.48	1.58	1.62	1.67
$4 \times 4 \times \frac{5}{16}$ $\frac{3}{8}$ $\frac{1}{2}$	4.81	1.23	1.65	1.73	1.78	1.82
	5.72	1.22	1.66	1.74	1.79	1.83
	7.50	1.20	1.68	1.77	1.82	1.86
$5 \times 5 \times \frac{3}{16}$ $\frac{1}{2}$ $\frac{5}{8}$	7.22	1.54	2.06	2.15	2.19	2.24
	9.50	1.52	2.08	2.17	2.22	2.26
	11.72	1.51	2.11	2.19	2.24	2.29
$6 \times 6 \times \frac{1}{2}$ $\frac{5}{8}$ $\frac{3}{4}$	11.50	1.84	2.48	2.56	2.61	2.65
	14.22	1.83	2.50	2.59	2.63	2.68
	16.88	1.82	2.53	2.62	2.66	2.71

TABLE 16

RIVET AND BOLT VALUES (B.S. 449—RATIOS)

The working axial tensile stress on net areas of sections, in tons per sq. in. = F_t . Then, in terms of F_t the rivet and bolt values in single shear (F_s), double shear (F_{ds}), bearing (F_b) and axial tension (F_a) are :—

	F_s	F_{ds}	F_b	F_a
Shop rivets and T.F. turned bolts .	$2/3F_t$	$4/3F_t$	$4/3F_t$	$5/9F_t^*$
Site rivets	$5/9F_t$	$10/9F_t$	$10/9F_t$	$4/9F_t$
Black bolts	$4/9F_t$	$8/9F_t$	$8/9F_t$	$5/9F_t^*$

* On bolts less than 3/4 in. dia. On all bolts 3/4 in. dia. and up $F_a = 6/9F_t$. The axial stress is on the area at the bottom of the bolt thread.

SHOP RIVETS AND TIGHT-FITTING TURNED BOLTS

Dia. of hole, in.	Area in sq. in.	Shear value in tons per rivet.		Bearing value in tons per rivet or bolt for a plate thickness (in inches) of										Axial tension in tons		
		s'gle	d'ble	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	Value of one Rivet	Bolt*		
1/2	5/8	·196	1·05	2·09	1·33	1·67	2·00	2·33	2·67	$F_t = 8^T.$					0·87	0·54
		·307	1·64	3·27	1·67	2·08	2·50	2·92	3·33					1·36	0·90	
3/4	13/16	·442	2·36	4·71	2·00	2·50	3·00	3·50	4·00	4·50	5·00			1·96	1·62	
		·519	2·77	5·53	2·17	2·71	3·25	3·79	4·33	4·87	5·42	5·97		2·30	1·96	
7/8	15/16	·601	3·21	6·41	2·33	2·92	3·50	4·08	4·67	5·25	5·83	6·41		2·67	2·25	
		·690	3·68	7·36	2·50	3·13	3·75	4·38	5·00	5·62	6·25	6·87	7·50	3·06	2·65	
1		·785	4·19	8·38	2·67	3·33	4·00	4·67	5·33	6·00	6·67	7·33	8·00	3·49	2·96	

1/2	·196	1·18	2·36	1·50	1·88	2·25	2·63	3·00	$F_t = 9^T.$					0·98	0·61
5/8	·307	1·84	3·68	1·88	2·34	2·81	3·28	3·75						1·53	1·02
3/4	·442	2·65	5·30	2·25	2·81	3·38	3·94	4·50	5·06	5·63	6·70			2·21	1·82
13/16	·519	3·11	6·23	2·44	3·05	3·66	4·27	4·88	5·48	6·10				2·59	2·20
7/8	·601	3·61	7·21	2·63	3·28	3·94	4·59	5·25	5·91	6·56	7·22	8·44		3·01	2·53
15/16	·690	4·14	8·28	2·81	3·52	4·22	4·92	5·63	6·32	7·03	7·73			3·45	2·98
1	·785	4·71	9·42	3·00	3·75	4·50	5·25	6·00	6·75	7·50	8·25	9·00		3·93	3·32

1/2	·196	1·31	2·62	1·67	2·08	2·50	2·92	3·33	$F_t = 10^T.$					1·09	0·67
5/8	·307	2·04	4·08	2·08	2·60	3·12	3·64	4·17						1·71	1·12
3/4	·442	2·94	5·88	2·50	3·12	3·75	4·38	5·00	5·62	6·25	7·46			2·46	2·02
13/16	·519	3·46	6·92	2·71	3·38	4·06	4·74	5·42	6·09	6·77				2·88	2·44
7/8	·601	4·01	8·02	2·92	3·65	4·38	5·10	5·83	6·56	7·29	8·00	9·38		3·34	2·81
15/16	·690	4·45	8·90	3·12	3·91	4·69	5·47	6·25	7·03	7·80	8·59			3·83	3·31
1	·785	5·24	10·5	3·33	4·17	5·00	5·83	6·66	7·50	8·33	9·17	10·00		4·37	3·69

SITE RIVETS. Find the number of shop rivets required and increase this number by 20 per cent. for shear and bearing, and by 25 per cent. for axial tension.

BLACK BOLTS. Find the number of turned bolts required and increase this number by 50 per cent. for shear and bearing. Black and turned bolts have the same values for axial tension.

TABLE 17
PITCH MULTIPLICATION TABLE

The heading gives the pitch in inches, the other figures the total distance in feet and inches

No. of	2½	2½	2½	2½	2½	2½	3	3½	3½	3½	3½	4	4½	4½	4½
2	4½	4½	5½	5½	5½	5½	6	6½	6½	7	7½	8	8½	9	9½
3	6½	7½	7½	7½	8½	8½	9	9½	10½	10½	11½	11½	11½	12½	12½
4	8½	9½	10½	10½	11½	11½	11	11½	12½	12½	13½	13½	13½	14½	14½
5	10½	11½	12½	12½	13½	13½	13	13½	14½	14½	15½	15½	15½	16½	16½
6	12½	13½	14½	14½	15½	15½	14	14½	15½	15½	16½	16½	16½	17½	17½
7	14½	15½	16½	16½	17½	17½	15	15½	16½	16½	17½	17½	17½	18½	18½
8	16½	17½	18½	18½	19½	19½	16	16½	17½	17½	18½	18½	18½	19½	19½
9	18½	19½	20½	20½	21½	21½	17	17½	18½	18½	19½	19½	19½	20½	20½
10	20½	21½	22½	22½	23½	23½	18	18½	19½	19½	20½	20½	20½	21½	21½
11	22½	23½	24½	24½	25½	25½	19	19½	20½	20½	21½	21½	21½	22½	22½
12	24½	25½	26½	26½	27½	27½	20	20½	21½	21½	22½	22½	22½	23½	23½
13	26½	27½	28½	28½	29½	29½	21	21½	22½	22½	23½	23½	23½	24½	24½
14	28½	29½	30½	30½	31½	31½	22	22½	23½	23½	24½	24½	24½	25½	25½
15	30½	31½	32½	32½	33½	33½	23	23½	24½	24½	25½	25½	25½	26½	26½
16	32½	33½	34½	34½	35½	35½	24	24½	25½	25½	26½	26½	26½	27½	27½
17	34½	35½	36½	36½	37½	37½	25	25½	26½	26½	27½	27½	27½	28½	28½
18	36½	37½	38½	38½	39½	39½	26	26½	27½	27½	28½	28½	28½	29½	29½
19	38½	39½	40½	40½	41½	41½	27	27½	28½	28½	29½	29½	29½	30½	30½
20	40½	41½	42½	42½	43½	43½	28	28½	29½	29½	30½	30½	30½	31½	31½
21	42½	43½	44½	44½	45½	45½	29	29½	30½	30½	31½	31½	31½	32½	32½
22	44½	45½	46½	46½	47½	47½	30	30½	31½	31½	32½	32½	32½	33½	33½
23	46½	47½	48½	48½	49½	49½	31	31½	32½	32½	33½	33½	33½	34½	34½
24	48½	49½	50½	50½	51½	51½	32	32½	33½	33½	34½	34½	34½	35½	35½

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